Development of Seismic Design Approach for Freestanding Freight Railroad Embankment Comprised of Lightweight Cellular Concrete

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ABSTRACT: Recent advances in research, laboratory testing and field evaluations of lightweight cellular concrete have led to an increased understanding about its application as a geomaterial. Recently, lightweight cellular concrete has been used to construct a 40-foot high by 50-foot wide freestanding railroad embankment with vertical sidewalls near Colton, California. The embankment and flyover structures are about 7,000 feet long and consist of 220,000 cubic yards of lightweight cellular concrete. The embankment was designed to support 3 simultaneous Cooper E-80 freight railroad live loads and seismic loading from a 2500-year return period earthquake event. In order to provide an earthquake resilient material, cellular concrete was selected because of its relatively low density (25 to 37 pounds per cubic foot) and high compressive strength (140 to 425 pounds per square inch), when compared with traditional backfill materials. This alternative also provided a reduced embankment footprint and corresponding dead load, which reduced foundation settlement and possible inertial interaction with nearby utilities and infrastructure.

Comprehensive seismic design guidance for lightweight cellular concrete embankments has not been fully developed in the U.S, but a rational approach has been developed for freestanding geofoam embankments. A similar approach was incorporated in the design process of the Colton, California embankment. This paper discusses the design process including: (1) selection and development of spectrum-compatible time histories for both horizontal and vertical components of strong ground motion; (2) development of design and evaluation methodologies; (3) detailed numerical evaluation using finite element [QUAKE/W] and finite difference techniques [FLAC 2D]; (4) assessment of the load-deformation characteristics of the embankment system under seismic ground motion and (5) assessment of benefit of shear keys and ground improvement on limiting basal sliding during a seismic event.

OVERVIEW

Union Pacific Railroad (UP) has a shared at-grade crossing with BNSF Railway in the City of Colton, San Bernardino County, California. The crossing of these tracks is referred to as the "Colton Crossing". This project provides grade separation for these two heavy traffic lines and increases the efficiency of traffic flow in this area. This grade separation, or "flyover", structure consists of a cellular concrete embankment and a series of bridges along the alignment of the UP tracks. The total length of the Colton Crossing Flyover Retaining Structure is about 7,000 feet with an embankment width of about 50 feet and a maximum height of about 40 feet. See Figure 1 below.

Embankment section. The flyover embankment structure supports two railroad tracks and a maintenance access road, which was planned with lateral clearance for a possible future third track. The embankment section consists of, in descending order: (1) 8.5-foot wide concrete ties with ballasted track section [12 inches ballast/18 inches subballast], (2) 3-foot thick upper layer of Class IV cellular concrete, (3) variable thickness of Class II cellular concrete, (4) 2.5-foot thick Class IV layer of cellular concrete with a 4-foot deep shear key embedded in the foundation soils (at higher embankment sections), and (5) vibro-replacement stone columns approximately 15 ft deep in the foundation soils.

Cellular concrete was selected for the embankment based upon its low density and relatively high compressive and shear strength when compared to earthen fill. Use of lightweight material reduces the bearing pressure and the static and dynamic lateral earth pressures in the bridge abutment areas. A typical section of the proposed embankment structure with ground improvements is shown in Figure 1. Cellular concrete can also be pumped long distances for placement within a small footprint for the on-site equipment, which is critical in a tight construction site between Interstate-10 and active railroad tracks.

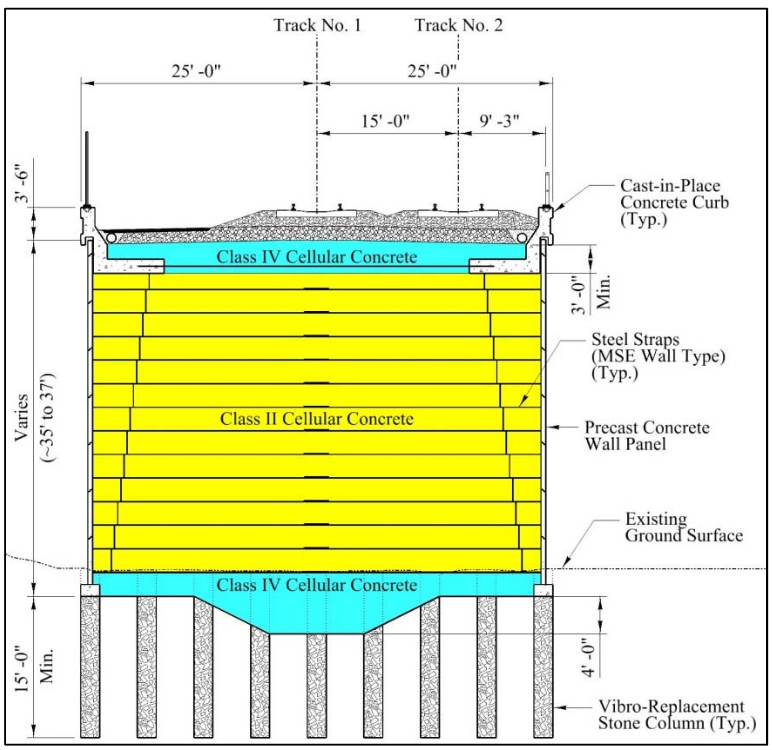

Figure 1. Typical flyover embankment section.

Cellular concrete. Cellular concrete is an engineered, low density material having a homogeneous cell structure formed by the addition of prepared foam or by the generation of gas within the fresh cementitious mixture. It is usually cast in densities ranging from about 20 to 120 pounds per cubic foot (pcf). The air cells created by the preformed foam may account for up to 80% of the total volume (Fouad, 2006).

Based upon research by Sandia National Laboratories (Sandia, 2004), the tensile to compression strength ratio is approximately 10% and the stress-strain relationship of unreinforced cellular concrete is similar to that of conventional concrete. Also, based upon available technical literature (Cellular Concrete LLC, 2011), the wet cast densities for Class II and Class IV cellular concrete are about 30 and 42 pounds per cubic foot (pcf), respectively; and the dry densities are about 25 and 37 pcf, respectively. The corresponding as-cast compressive strengths for these densities are about 40 and 120 pounds per square inch (psi), respectively; and the 28-day compressive strengths are about 140 and 412 psi, respectively.

Ground Improvement and Shear Key. Vibro-replacement stone columns were selected as the ground improvement method at the site to increase the relative density and shear strength of the subgrade and shallow foundation soils, and improve the overall stability of the embankment structure. The vibro-replacement technique utilizes a small mobile rig to insert a vibrating probe and construct stone columns of depths up to 30 feet below the ground surface. In addition to the ground improvement, a shallow shear key consisting of light-cellular concrete was bedded in the foundation soils to improve the sliding resistance of the embankment during the design basis earthquake (Figure 1). The influence of these improvements on the overall stability of the embankment structure will be discussed subsequently.

SITE CHARACTERIZATION

Existing subsurface conditions. Geotechnical investigations for this site were performed by C.H.J. Inc. (C.H.J.) during the summer of 2010. The conditions at this site consist of [top to bottom]: (1) thin layer of loose, silty sandy fill [1 to 9 feet thick], (2) loose to medium dense silty sand [5 to 25 feet thick], (3) medium dense silty sand [0 to 35 feet thick], (4) dense to very dense silty sand [thickness unknown, bottom of boreholes] (C.H.J., 2011). Groundwater was located between 117 and 123 feet below the ground surface as measured in the boreholes.

Shear wave velocity measurements were attempted at the site via the seismic cone penetrometer (SCPT). However, such measurements were not presented in the geotechnical report because of the excessive background noise caused by traffic on the adjacent freeway. The probabilistic seismic hazard analysis (PHSA) assumed that the site classified as Site Class D (stiff soil profile) with an estimated average shear wave velocity of 270 m/s in the upper 100 feet (C.H.J., 2011).

Seismic Hazard. The site is not located in a mapped fault rupture zone (C.H.J., 2011); however due to the presence of several nearby, active faults the expected ground motion at this site is large. The project geotechnical report provided acceleration response spectra for the three probabilistic design basis earthquakes (Figure 2). The Level 1, 2 and 3 events represent spectral accelerations having average return periods of 72, 475 and 2475 years, respectively. [Note: the calculated vertical design accelerations at this site are considerably higher than would be encountered in most other areas of California. For example, vertical Peak Ground Acceleration (PGA_v) is on the order of 90% of the corresponding horizontal value based on published attenuation relationships (Campbell-Bozorgnia, 2008)].

The controlling fault at this site for the Level 3 event is the San Jacinto fault zone (SJFZ) based on the seismic deaggregations of PGA and the 0.2 sec spectral acceleration values. Depending on the fault rupture scenario, the controlling earthquake is approximately M7.0 to M7.7 event. The San Jacinto fault is the closest known active fault to the site and is about 1.4 km northeast of the planned alignment (C.H.J., 2011). C.H.J. (2011) reports that the SJFZ is a system of northwest-trending, right-lateral, strike-slip faults that traverses the southwestern San Bernardino Valley and indicates a zone of active surface faults, tectonic deformation and folding.

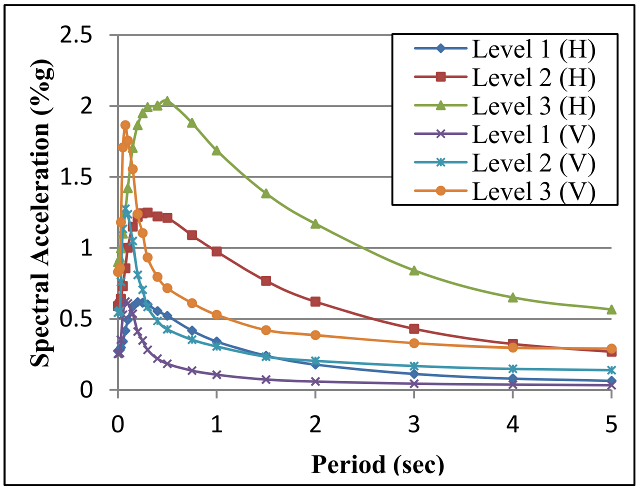

Figure 2. Design horizontal and vertical response spectra (5 percent damped).

DESIGN AND PERFORMANCE CRITERIA

Performance criteria established for the Flyover Retaining Structure adopted recommendations from AREMA (2010) for bridges, which include three levels of ground motion with corresponding performance goals as follows: (1) Level 1 – The embankment structure should remain intact with no permanent deformation (i.e. the seismic loads must remain within the elastic range of the stress-strain curve of the embankment); (2) Level 2 – The embankment structure should be repairable, with only minor permanent deformation; and (3) Level 3 – The embankment structure must not collapse after experiencing permanent deformations.

PRELIMINARY STABILITY ANALYSES

Comprehensive seismic design guidance for free standing lightweight concrete embankments has not been fully developed in the U.S, but a rational numerical modeling approach applicable to the seismic design and evaluation of freestanding geofoam embankments is discussed in Bartlett and Lawton (2008) and Bartlett et al. (2011). Principals from this approach were applied in the preliminary analyses.

Preliminary global stability analyses of potential deep-seated failure surfaces were performed using the computer program SLOPE/W (GEO-SLOPE, 2007). The SLOPE/W program uses limit equilibrium techniques to search for the critical failure surface (i.e., that surface with the minimum factor of safety). The inertial acceleration used in the analyses for the embankment corresponded to the spectral horizontal acceleration value computed at the fundamental period of the embankment, T_o. The corresponding inertial acceleration was not varied in the embankment because the light-weight cellular concrete embankment behaves more like a rigid body under elastic conditions. Various values of T_o were calculated along the alignment according to the methodologies discussed in Horvath (2004), Bartlett and Lawton (2008), and Bartlett et al. (2011) using the corresponding height, width, and elastic properties of the embankment at that location.

Ultimately, it was found that the highest embankment cross-section of about 40 feet controlled the design because this maximum height produced the highest inertial forces within the embankment and the lowest factors of safety against basal sliding. The corresponding T_o value for the controlling section is about 1.15 seconds. The corresponding design horizontal spectral accelerations for each AREMA level earthquake are shown in Figure 3. These values were used as the horizontal inertial acceleration in the limit equilibrium analyses.

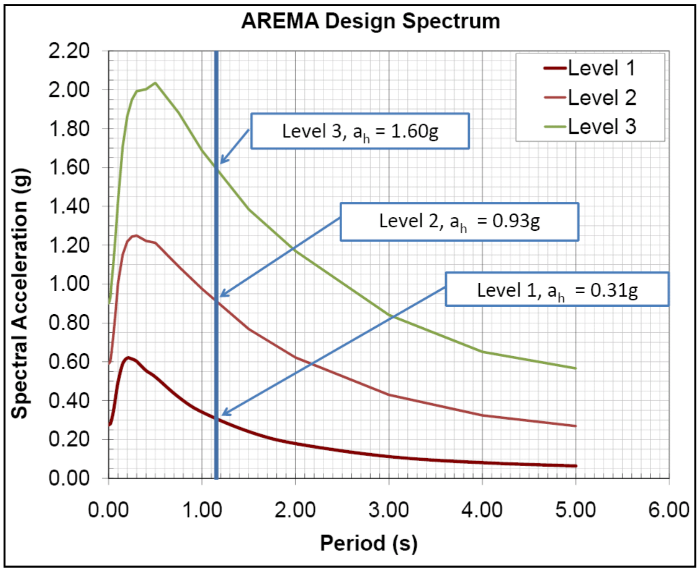

Figure 3. Horizontal Spectral Acceleration Values at T_o for the Design Basis Events.

For cases where the computed global stability factor of safety (FS) was below 1.0, a Newmark sliding block displacement analyses (Kramer, 1996) was also performed using SLOPE/W to provide preliminary estimates of the permanent deformation for each of the AREMA earthquake events. Using the computed accelerations at T_o for the AREMA Level 1 event (Figure 3), the computed minimum FS was 2.0. This high factor of safety suggests no yielding will take place within the embankment structure or the foundation soils. Similarly, for the AREMA Level 2 and 3 earthquake events, the computed FS were 0.7 and 0.4, respectively, which suggest yielding, predominately in the foundation soils. The resulting permanent displacements were estimated as 1 and 7 inches for the AREMA Level 2 and 3 events, respectively.

In addition to these simplified limit equilibrium analyses, the global seismic stability of the embankment structure was also investigated using the finite element method and computed stress time history calculated from QUAKE/W (GEO-SLOPE, 2007) for the controlling earthquake record. These latter results were found to agree very closely with the limit equilibrium global stability analyses using the spectral accelerations from Figure 3.

DEVELOPMENT OF STRONG GROUND MOTION

More elaborate deformation analyses were also conducted that required horizontal and vertical acceleration time histories for the design basis events. These were developed as spectrum compatible time histories using spectral matching techniques from the computer program RSPMATCH (Abrahamson, 1998). The goal of spectral matching is to generate a set of realistic time histories that satisfy seismological and geological conditions appropriate for the seismic source and site conditions at the candidate site. The main considerations for selecting time histories are: (1)

appropriate earthquake magnitude, (2) faulting mechanism (e.g., strike-slip, vs. dip-slip, etc.), (3) tectonic regime, (4) source-to-site distance, and (5) geological structure. The candidate records should be selected from earthquake events that have similar conditions, whenever possible.

Selection of Time Histories. Candidate strong motion records were selected from the Pacific Earthquake Engineering Research (PEER) Center Strong Motion website based on earthquake magnitude, source distance and faulting style that was similar to the candidate site. In addition to the spectral shape and amplitude, selection of time histories should consider the dependence of the system response on time domain characteristics such as earthquake duration, pulse shape, pulse sequencing, etc. AREMA (2010) does not specify the number of time histories required for site-specific, time-domain, nonlinear analysis. However, it is recommended that at least 7 independent time histories be used for such analyses.

In addition, we recommend the candidate acceleration time histories should be obtained from rock or very stiff soil sites (Site Class B or C), whenever possible, and should be statistically independent motions (i.e., should have no statistical or spatial correlation). Synthetically generated time histories are not recommended for ground response analyses because such records may not have near field and other site effects, which may be important for non-linear time domain analyses of nearby, large earthquakes.

The time histories selected for our analyses were not modified for duration. This was deemed unnecessary, because the selected time histories have approximately the same earthquake magnitude and distance from the seismic source distance as the controlling earthquake at the Colton Crossing site.

Spectral Matching. Most acceleration time histories, when taken at face value without modification, do not provide an adequate match to the design spectrum, thus they must be scaled, adjusted, or matched to provide a better fit. Spectral matching may be done in either the time domain or the frequency domain in such a way that the spectral acceleration values of the spectrally matched time history match a target response spectrum within a prescribed tolerance. The spectral matching

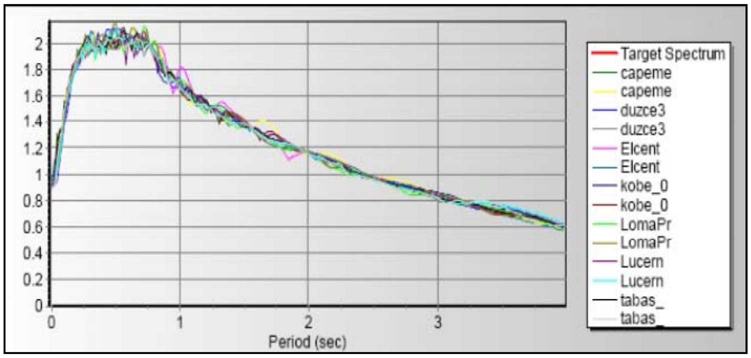

Figure 4. Spectrally matched time histories for Level 3 horizontal response spectra.

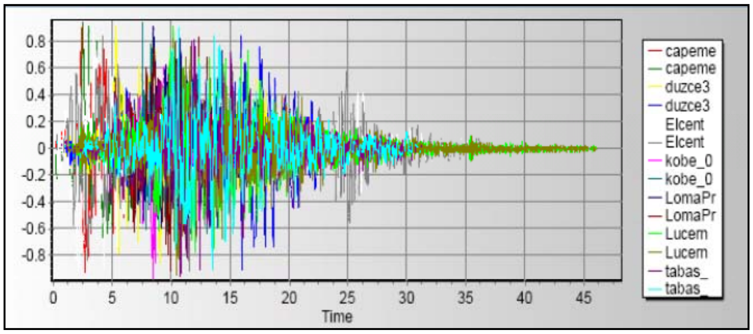

Figure 5. Comparison of spectrally-matched time histories with Level 3 target spectrum.

performed by RSPMATCH for this project was done throughout the full spectral range with 5 percent error tolerance. For example, Figure 4 shows the horizontal response spectral results from RSPMATCH, and the corresponding spectrally-matched time histories are shown in Figure 5. Vertical time histories used in the load-deformation analyses discussed below followed the same process.

Baseline Correction. A baseline correction should be performed on the input time histories after spectral matching. The spectral matching process may introduce some drift into the processed record, which must be corrected. Seismosignal (http://www.seismosoft.com/en/SeismoSignal.aspx) was used to baseline correct the spectrally-matched time histories.

Deconvolution. The spectrally-matched, strong motion records were deconvolved to a depth equal to the base of the 2D numerical model using the 1D equivalent linear procedures described by Mejia and Dawson (2006). The steps and boundary conditions required to convolve the motion upward through the 2D model are described later. The deconvolution analysis was done with PROSHAKE™ using linear-elastic soil properties for the foundation soils. (Preliminary analyses showed that if nonlinear soil properties were assigned to the deconvolution model, then the deconvolution analysis produced numerical instability due to the large amplitude of the input motion.) Thus, the foundation soils in the model were treated as linearly elastic materials in both the deconvolution and subsequent convolution analyses. The deconvolution motion was convolved back to the surface to verify that the deconvolution-convolution analyses were capable of reproducing the original spectrally-matched time history at the ground surface. This step ensured that the design basis ground motion was successfully delivered to the base of the embankment without any amplification or attenuation of the spectral values. However, the drawback to this procedure is that potential inelastic deformation of the foundation materials could not be estimated.

FINAL SEISMIC STABILITY AND LOAD-DEFORMATION ANALYSES

The computer programs QUAKE/W (Geostudio, 2007) and FLAC 2D (Itasca, 2005) were used to perform the seismic stability and load-deformation analyses of the embankment. The finite element option in QUAKE/W was used to model the embankment and foundation soil elastically, with no basal sliding interface, thus maximizing the seismic stresses induced in the embankment structure. The computer program FLAC was used to analyze the potential for yielding within the embankment, basal sliding, uplift and rocking of the embankment and to model the performance of the proposed ground improvement in the foundation soils.

QUAKE/W Equivalent Linear Elastic Analyses. In QUAKE/W, the Direct Integration Method is used to compute the motion and predict excess pore-water pressures, if groundwater were present, resulting from inertial forces at user-defined time steps. Tables 1a and 1b present the maximum computed accelerations and stresses at various "history" points of interest (points A through G), for all of the design strong ground motions described previously. The history points were taken at the following locations within the embankment structure (see Figure 6):

- Point A - "Quiet" point located at the ground surface far away from the embankment (used to check the computed ground acceleration and verify that the record has been deconvolved accurately);

- Point B – Top of embankment at centerline (compare this to computed fundamental period of the embankment);
- Point C – Center of Class II cellular concrete layer at the centerline of the embankment;
- Point D – Class II and Class IV cellular concrete interface at the centerline of embankment;
- Point E – Bottom of class IV cellular concrete shear key at the centerline of embankment;
- Point F – Precast panel/ footing interface; and
- Point G – Bottom of panel footing at soil interface

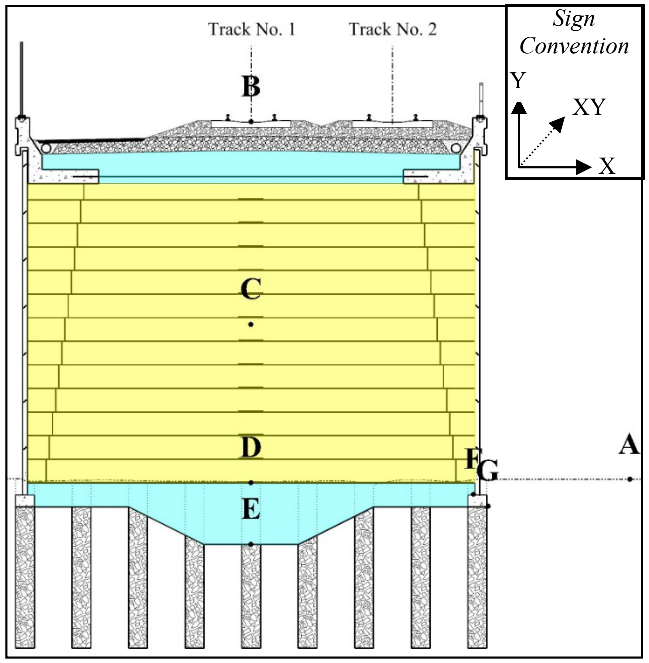

Figure 6. History Point Definition Schematic

Table 1a – Summary of Seismic Accelerations and Stresses at Level 2 Event

	History Point						
	A	B	C	D	E	F	G
X-Acceleration (%g)	0.85	0.85	0.72	0.66	0.7	0.68	0.67
Y-Acceleration (%g)	0.96	0.32	0.30	0.28	0.28	0.56	0.55
XY-Shear Stress (psf)	245	159	948	1,198	1,147	2,995	4,817
X-Stress (psf)	498	243	162	492	790	5,991	6,785
Y-Stress (psf)	572	230	1,198	1,790	2,200	2,749	8,597

Table 1b – Summary of Seismic Accelerations and Stresses at Level 3 Event

	History Point						
	A	B	C	D	E	F	G
X-Acceleration (%g)	1.72	1.48	1.19	0.98	0.97	0.97	0.97
Y-Acceleration (%g)	1.50	0.50	0.47	0.44	0.44	0.92	0.90
XY-Shear Stress (psf)	495	273	1,607	1,984	1,789	4,590	7,315
X-Stress (psf)	693	275	196	816	1,113	9,296	10,568
Y-Stress (psf)	712	263	1,382	2,058	2,518	3,460	12,404

According to AREMA (2010), the maximum useable strain at the extreme concrete compression fiber is equal to 0.003 in/in at concrete crushing, which corresponds to the ultimate load or extreme event. The maximum shear strength of unreinforced cellular concrete was assumed to be, $V_c = 0.75 \times 2 \times \sqrt{f'_c}$ per ACI 213R-87 (ACI, 1999). Based upon these criteria, the maximum shear stress for the Class II and Class IV cellular concrete are 2,556 and 4,384 psf, respectively. Therefore, for the estimated stresses at the history points listed in Tables 1a and 1b, the embankment should remain elastic during and following the AREMA Level 2 and Level 3 earthquakes. It should be noted that history points F and G were located near the concrete panel/footing interface and the maximum shear stress of 18,215 psf was assumed based upon 4,000 psi compressive strength concrete for this material.

The ultimate compressive stresses (Y-Stress) of Class II and Class IV cellular concrete are 20,160 psf (140 psi) and 59,328 psf (412 psi), respectively as discussed previously. Neither the AREMA Level 2 or Level 3 earthquake analyses produced compressive stresses in excess of the cellular concrete's allowable compressive strength, which was taken as 30% of the ultimate compressive strength ($0.3*f'_c$) based on AREMA Section 2.26.1 for bearing on a loading area.

FLAC Non-Linear Analyses. The finite difference method is employed by FLAC (Itasca, 2005) which allowed the investigation of other potential failure mechanisms beyond those analyzed in QUAKE/W. The primary advantage that FLAC offered was its interface nodes, which allowed for sliding and separation between dissimilar materials. FLAC was used to assess: 1) internal shear and tensile failure within the embankment; 2) basal sliding at the embankment/foundation interface; 3) excessive rocking/overturning; and 4) dynamic bearing capacity failure of the improved foundation for the AREMA Level 2 and 3 earthquake events.

The results of the FLAC analyses indicate that the cellular concrete embankment and shear key remain within the elastic range in compression, tension and shear for all Level 2 and 3 earthquake time histories. In addition, the FLAC analyses suggest that only a minor amount of permanent basal sliding is expected (i.e., 4 and 6 inches maximum for the Level 2 and 3 earthquakes, respectively). These same analyses also indicate that rocking and uplift are not important failure modes, because no significant uplift is occurring at the basal corners and these areas are not being damaged (i.e., not yielding). However, the FLAC analyses show that the underlying ground improvement does not remain in the elastic range, but will experience some yielding, which produces about 6 inches of horizontal displacement at the embankment/treated soil interface for the Level 3 earthquake.

Comparison of QUAKE/W and FLAC models. The results of the QUAKE/W and FLAC models were found to be very complimentary and similar to each other. Both models were able to convolve the design ground motions to the surface accurately. In addition, the results of both models indicate some deformation of the shear key at its interface with the soil. Also, as previously discussed, the QUAKE/W model was found to be very useful in estimating conservative seismic accelerations and internal forces at various points throughout the embankment and soil structure. The FLAC model was very useful in evaluating the permanent deformation of the embankment and its interface with the foundation.

FINDINGS AND CONCLUSIONS

With the incorporation of the proposed ground improvement and light weight cellular concrete shear key, seismic global stability analyses indicate that the cellular concrete embankment will remain stable under AREMA Level 1 seismic loading (F.S. > 1.0), and the estimated permanent displacement of the highest embankment structure section is expected to range from 1 to 4 inches at the Level 2 earthquake, and from 4 to 7 inches at the Level 3 earthquake.

Seismic load-deformation analyses indicate that the cellular concrete embankment will not yield under any of the AREMA Level 1, 2 and 3 earthquakes, and its response should remain elastic. Also, these analyses also indicate that the shear key is integral to limiting the basal sliding of the embankment structure and is an important design feature. Additionally, the vibro-replacement columns also help in limiting the sliding and deformation of the foundation soil in a secondary role compared to the benefit gained by the shear key.

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