DESIGN, CONSTRUCTION, AND MONITORING OF A 14.9M HIGH GEOSYNTHETIC REINFORCED SEGMENTAL RETAINING WALL IN A SEISMICALLY ACTIVE REGION

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ABSTRACT

Development of a new master planned community in a seismically active area required a 14.9 m high retaining wall to be constructed. The segmental retaining wall option was chosen based on economics, while the installation contractor and materials incorporated in the wall were chosen for their past performance and familiarity to the local regulatory agency. The design was developed using National Concrete Masonry (NCMA) methods. Construction procedures and details of monitoring are discussed. Post construction movements are provided for vertical and horizontal wall deformations. In addition, movements resulting from the October 1999 Hector Mine earthquake are documented. Monitoring of the wall will continue in anticipation of capturing the effect of additional seismic events.

1. INTRODUCTION

Ladera Ranch, a new, mastered planned community in Southern California is currently being developed. The Ladera Ranch development is located several miles east of the well known City of Mission Viejo in the Santa Ana Mountain foothills of Orange County, California. The new community will be approximately 16 square kilometers in size and contain over 8500 homes along with various commercial, recreation and school sites.

Construction of a new community requires development of infrastructure facilities including access roadways. The primary access to Ladera Ranch includes extension of Antonio Parkway, a six-lane arterial roadway which crosses a ravine near the northern entrance to the community. Plans to facilitate crossing the ravine incorporated a bridge along with an originally planned 2:1 slope transitioning from roadway grades down into the existing ravine. However, during the design process for the planned roadway, alignment adjustments were required to accommodate an existing County of Orange storm drain outlet (constructed during an earlier grading phase of the Antonio Parkway extension project) which limited the horizontal distance into which a slope could be fit.

Consideration of several grade transitioning systems including walls and slopes were evaluated. While being cost effective, steepened, reinforced soil slopes were ruled out due to space constraints and lack of approval precedent from the regulatory agency; the County of Orange. Given the height of the retained section, a conventional cantilever wall was ruled out due to cost (estimated at $1.5 million), aesthetics and construction time. Ultimately, a 248 m long by up to14.9 m high (1840 m2) reinforced segmental retaining wall (SRW) was chosen to satisfy the grade transition problem while saving nearly $1,000,000.

2. GEO-ENVIRONMENT CONDITIONS

2.1 Geotechnical Conditions and Earth Material Properties

Subsurface materials at the site proposed for the SRW generally consist of Miocene aged Monterey Formation bedrock overlain by late Quaternary alluvial terrace deposits. The bedrock typically consists of stiff, unoxidized, siltstones and claystones with geologic structure favorable to the proposed wall orientation. The sedimentary bedrock is comprised of low to high plasticity silts and clays with approximately 30 to 40 percent silt and 20 to 50 percent clay size particles. Liquid limit (LL) and Plasticity Index (PI) values range from 40 to 70 and 30 to 50, respectively. Drained effective stress strength parameters (ASTM D-3080) representative of peak conditions were determined to be c’p = 54 kN/m2, Ø’p = 26 degrees.

The alluvial terrace deposits consist of dense to very dense low plasticity clayey sands and sandy clays (SC to CL). The terrace materials contain approximately 30 to 55 percent silt and clay size particles (Passing the No. 200 US Std. Sieve) with LL and PI values ranging from 30 to 40 and 20 to 30, respectively. Shear strengths representative of the in-situ and remolded terrace deposits (no significant difference between the in-situ and remolded conditions) for peak conditions are c’p = 23.9 kN/m2, Ø’p = 32 degrees. Remolded samples were compacted to 90% of maximum density (ASTM D-1557).

Groundwater was not encountered or anticipated to exist within the zone which might influence the proposed SRW over its design life. Geotechnical data at the wall location was extrapolated from borings drilled in the vicinity of the wall (Goffman, McCormick & Urban, Inc., 1996), from observations and testing during preliminary grading of Antonio Parkway (Goffman, McCormick & Urban, Inc., 1999), and from instrumentation borings drilled following construction of the wall.

FIGURE 1: General Location Map Relative To Faults

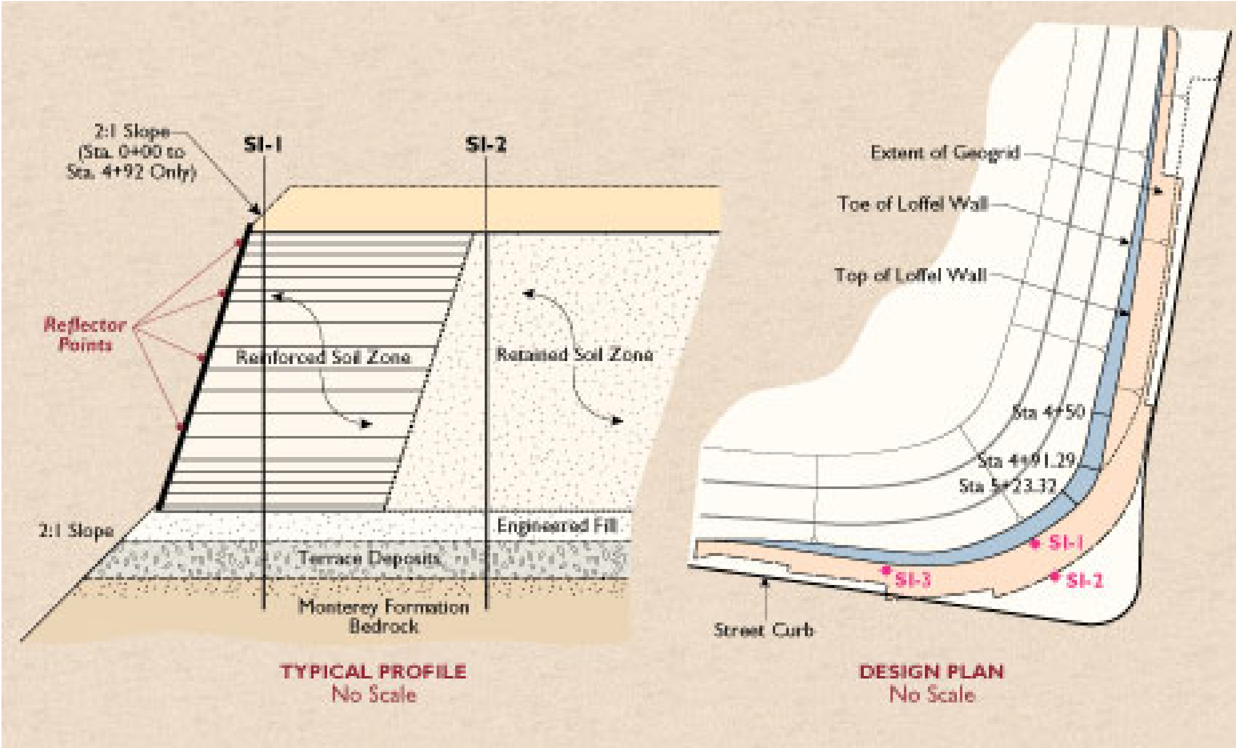

FIGURE 2: Design Plan and Profile

2.2 Seismic Environment

Most of Southern California is subject to some level of ground shaking (ground motion) as a result of movement along active and potentially active fault zones in the region. The nearest seismogenic faults recognized by the California Division of Mines and Geology (CDMG 1999) are the offshore segment of the Newport-Inglewood fault and the Elsinore fault (Jennings, C.W., 1994). These faults are located approximately 15 kilometers southwest and 25 kilometers northeast of the project site, respectively (Figure 1).

Preliminary research by some scientists at the Southern California Earthquake Center (SCEC) suggest the project site may also be located near or underlain by segments of a low-angle fault system (e.g., blind thrusts), the fault surfaces of which do not necessarily break the ground surface during sizeable earthquakes. This fault system, known as the San Joaquin Hills Blind Thrust, has not yet been studied with enough detail to determine the existence, location, or subsurface geometry of the fault let alone classify it as "Active" pursuant to the guidelines of the Alquist-Priolo Earthquake Fault Zoning Act (Hart and Bryant, 1997).

Several sizeable, historic earthquakes have occurred in southern California resulting in ground motions at the site and surrounding area, most recently the October 16, 1999 MW 7.1 Hector Mine Earthquake. Peak horizontal ground accelerations (PHGA) at the site from the Hector mine event were estimated from data available through the California Strong Motion Instrumentation Program (CSMIP). The nearest CSMIP station to the site is Capistrano Beach - 15/Via California Bridge, Station 13795, located approximately 5 kilometers from the site. Review of the preliminary CSMIP data indicate that peak horizontal ground acceleration at Station 13795 was approximately 0.03g. CSMIP stations northwest and east of the site area recorded similar accelerations. Peak acceleration at the bridge bent for Station 13795 was approximately 0.1g. Given these data, PHGA at the project site were approximately 0.03g, while peak accelerations at the crest of the SRW wall were most likely on the order of 0.1g.

The level of potential ground motion at a given site resulting from an earthquake is a function of several factors including earthquake magnitude, type of faulting, rupture propagation path, distance from the epicenter, earthquake depth, duration of shaking, site topography, and site geology. Review of the available literature indicates that a peak horizontal ground acceleration of approximately 0.3 g for 475-year annual return period (Petersen et al., 1996) would be appropriate for the site. The aforementioned return period corresponds to that required by the UBC for the design of structures.

3. DISCUSSION OF SRW CONTRACTOR AND MATERIALS

The selection process for the wall type, construction contractor and materials to be incorporated in the wall was undertaken with strong consideration for the rigorous regulatory review and permitting process as well as time sensitive, critical path posed by the roadway supporting wall structures under consideration.

3.1 SRW Blocks and Geogrid Reinforcement

The materials chosen for wall design included a trough-shaped, plantable segmental block supplied by Soil Retention Systems, polyester fiber XT geogrids as supplied by TC Mirafi, and on-site alluvial terrace soils to be used in the reinforced and retained wall zones. The polymeric reinforcement materials selected for use in the proposed walls were chosen based on their past performance on similarly sized local projects as well as familiarity to the regulatory agency reviewing the project. In addition, the high tensile strength range of the geogrid, stable, large dimension of the trough shaped plantable block, and local availability (dependable US supply for geogrid and S. CA supply for block) also contributed to the selection of the wall system. Properties of the materials used for wall construction are summarized in the following table.

3.2 Wall Building Contractor

The wall building contractor, Soil Retention Systems, was selected based on his on-time delivery of turnkey construction for time-critical structures on many high-end commercial projects and experience with fully plantable (and hence aesthetically desirable) wall systems. Also considered in the selection process was the contractors unique installation process which is fully integrated with the grading activities and failure-free history.

BLOCK	TEST METHOD, UNITS	VALUE
Dimensions	Measured, cm (in)	-
• Length • Width • Height	- - -	50 (19.7) 45 (17.7) 25 (9.8)
• Weight	Kg (lbs)	Approximately 56.7 (125)
• Material • Compressive Strength	- ASTM C-140 psi @ 28 days	Pre-cast Concrete 4,000

GEOGRID	TEST METHOD	VALUE	
TYPE	- -	Type I - 10XT	Type II - 18XT
Polymer	MD x CMD	PET x PET	PET x PET
Coating Type	-	PVC	PVC
Tensile Strength (Ult)	ASTM D-4595, kN/m (lbs/ft)	115.3 (7,900)	136.6 (9,360)
Creep Reduced Strength	ASTM D-5262, kN/m (lbs/ft)	69.2 (4,740)	81.8 (5,605)
Long Term Design Strength	GRI-GG4, kNm (lbs/ft)	57.2 (3,917)	67.6 (4,632)
Reduced LTDS Required By County of Orange	LTDS/1.5, kN/m (lbs/ft)	38.1 (2,611)	45.1 (3,088)

TABLE 1: Properties of Materials Used in Construction of Segmental Retaining Wall

4. WALL DESIGN

The Ladera SRW was designed in accordance with the National Concrete Masonry Association (NCMA) design methodology (Collin, et al, 1997, and Bathurst, 1998). The NCMA method was selected for design because of its ability to incorporate seismic loads and familiarity to the regulatory agency (i.e. County of Orange).

The NCMA design method utilizes Coulomb Earth pressure theory and requires calculation of external, internal and global safety factors. The NCMA computer program (SRWall 2.1) incorporates geometric parameters including but not limited to block dimensions, wall slope/configuration, soil strength parameters, geogrid strength parameters (including geogrid to soil, geogrid to block interface shear) and shearing/sliding block to block parameters to calculate static safety factors. Conventional traffic surcharge loads from the adjacent roadway (i.e. Antonio Parkway) were modeled using distributed line loads. Global stability was evaluated using slope stability program PCSTABL (USDOT, 1986) assuming two failure modes: 1) rotational failure beneath the wall and 2) translational failure along the soil-geogrid interface.

Seismic wall analyses were conducted using the NCMA seismic procedures. The NCMA seismic design procedures incorporate horizontal seismic acceleration coefficients (in percent gravity). The selected design coefficient corresponded to a seismic event with a 475 year return period (ie. Having a 10 percent probability of exceedance in 50 years reduced by a factor of 1/3 as allowed by the County of Orange).

Although not supported in the literature for accelerations < 0.35g (Bathurst, R.J. and Cai, Z. 1995; and Ling, H. and Leschinsky, 1998), the County of Orange required incorporation of a vertical seismic acceleration coefficient. The vertical acceleration dictated by the County had to be equivalent to a value of at least two-thirds of the horizontal coefficient. Both the horizontal and vertical coefficients were utilized in determining the horizontal dynamic force via the Monobe-Okabe equation. Given these requirements and the seismic environment, a horizontal coefficient of 0.2g and a vertical coefficient of 1.3g were used in the design. Seismic design analysis also included performing a pseudo-static slope stability analysis using a regulatory agency mandated minimum seismic coefficient of 0.15g. Use of a 0.15g seismic coefficient in combination with a safety factor of 1.1 has resulted in satisfactory local dynamic performance of most slopes. Due to the stringent seismic requirements, seismic loading governed the final reinforcement layout and dimensioning of the soil reinforcement components.

The final SRW design incorporated segmental facing block units inclined at 20 degrees from vertical, reinforcing geogrid consisting of Type 1 and Type II geogrids (Table 1), drainage fill and leveling pad consisting of Caltrans Class II Permeable base (State of California Dept of Transportation 1992) and fill soil composed of previously described on-site terrace deposits (for both the reinforced and retained zones) (See Figure 2). The total wall height ranges from 5 to 14.9 m. An approximately 2 (horizontal) to 1 (vertical) slope descends from the toe of the wall along the full 248 m length. The wall toe embedment depth at the maximum height is approximately 1.8 m which renders a corresponding exposed height of roughly 13.1 m. The backfill above the wall crest varies in slope inclination from 0 (i.e. level) to 27 degrees (maximum height of 3.2 m). The geogrids are attached to the facing blocks via a wrap-around-pipe-in groove type connection and is set into the top of the facing block. The design geogrids have lengths ranging from 3 to 9.1 m and are spaced vertically at distances ranging from 0.3 to 0.61 m. A total of 23 reinforcement layers were utilized in the maximum wall section.

5. CONSTRUCTION PROCESS

Construction of the 14.9 m maximum height by approximate 248 m long wall was coordinated between the General Grading Contractor, ACI (responsible for fill placement in the adjacent roadway) and the wall building contractor, SRS. The coordination eliminated the back cut and double dirt handling problems and costs associated with typical SRW wall construction projects. The GC’s responsibility to the wall contractor was to deliver fill to the wall site as the adjacent roadway grades were raised. The wall builder’s responsibility was to handle the soil reinforcement facing blocks, drainage materials and to spread and compact the fill within the reinforced soil wall zone.

Facing blocks were removed from pallets and placed along the wall alignment using a mechanical "grapple" and subsequently adjusted into final resting position by hand. Structural fill was selectively obtained from cut areas in the vicinity of the project site, delivered to the wall area with 27 m3 capacity CAT 657 scrapers, moisture conditioned with water trucks as necessary, and spread into place by motor graders. CAT563 sheepsfoot rollers in conjunction with the wheeled soil delivery equipment were used for compaction. Compaction and soil delivery equipment was operated to within 0.6 m of the facia blocks. Materials within 0.6 m of the facing units were compacted with hand operated equipment. An automated process of filling of the trough shaped SRW units included use of a side dump bucket to spread a windrow of soil along the facing unit alignment. Subsequent to filling, any overfill was leveled and smoothed via hand screed. Pre-cut pieces of geosynthetic reinforcement were placed at the planned elevations and connected to the facing blocks as previously described above.

Construction of the wall progressed over an approximate three-week period during which time about 45,870 m3 of soil was moved into the wall area with production rates of up to 167 m2 of wall facing per day being achieved. Quality control and construction monitoring services for the wall building phases of construction were provided by Goffman, McCormick & Urban, Inc. of Rancho Santa Margarita. Construction monitoring requirements are noted in Table 2.

ITEM	REQUIREMENT	COMMENT
Block Placement	Per Plan	-
Geosynthetic Placement	Per Plan	-
Fill Strength (peak)	32 Degrees, 500 psf Cohesion	ASTM D-3080
Soil Density	90% at >2% over OMC*	ASTM D-1557
Fill Gradation	Oversize <10 cm	U.S. Standard Sieve

OMC* = Optimum Moisture Content

TABLE 2: Construction Monitoring Requirements

6. INSTRUMENTATION

Monitoring of post-construction movements incorporated arrays of survey reflector points embedded in the facing blocks along with slope inclinometers placed behind the wall face and reinforced zone.

6.1 Reflector Points and Movement Survey

Huitt-Zollars, a civil engineering company with licensed surveyors, was chosen to perform the monitoring program. During construction of the wall, surveys were performed at the face of the wall in order to verify the design wall batter of approximately 20 degrees.

In order to document the post-construction movement of the reinforced SRW, survey reflector point arrays were designed which would be monitored by licensed surveyors. Three vertical arrays of reflector points were set up at three different wall sections. Each array consisted of four points embedded into the block face (Figures 2 and 3) and equally spaced from the toe to the top of the wall. Both horizontal and vertical movements were monitored at three locations along the wall alignment. The three stations chosen included 4+50 (2:1 slope at crest of wall), 4+91.29 (midpoint between 2 other monitoring points), and 5+23.32 (maximum wall height). Table 3 provides the elevation and stationing of each survey reflector.

Two control stations and one backsite were installed in the vicinity of the toe of the wall and top of wall, respectively, and positioned so that surveyors would be able to monitor each of the reflector points. Readings were initiated on December 7, 1998. Reading intervals started at once a week for the first month, with intervals increased to once a month thereafter.

REFLECTOR POSTION	STATIONS		
	4+50 (Wall Height = 13.4 m) (B.O.W. Elevation = 169.7 m)	4+91.29 (Wall Height = 13.4 m) (B.O.W. Elevation = 171.0 m)	5+23.32 (Wall Height = 14.9 m) (B.O.W. Elevation = 172.0 m)
Highest Point	12[1] (~182.9m)[2]	10[1] (~184.5 m)[2]	11[1] (~186.6 m)[2]
2nd Highest Point	1[1] (~179.9 m)[2]	7[1] (~180.8 m)[2]	8[1] (~182.3 m)[2]
2nd Lowest Point	2[1] (~175.9 m)[2]	5[1] (~177.4 m)[2]	6[1] (~178.7 m)[2]
Lowest Point	3[1] (~173.2 m)[2]	4[1] (~175.9 m)[2]	9[1] (~175.3 m)[2]

[1] Reflector Point Designation
[2] Elevation of Reflector Point

TABLE 3: Summary of Survey Relfector Elevations and Stations

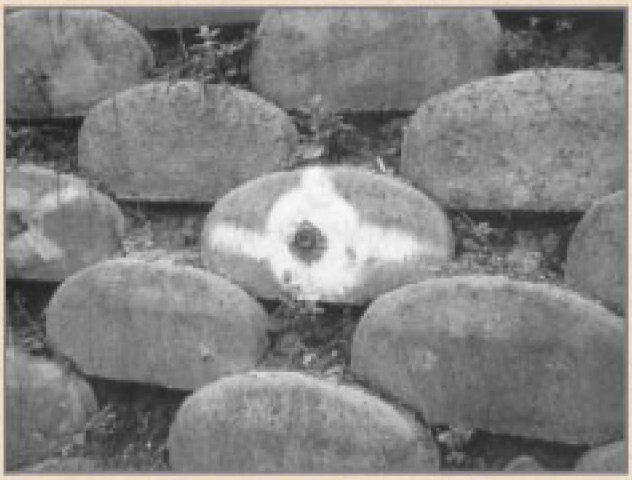

FIGURE 3:
Photograph of Survey Refelctor Point

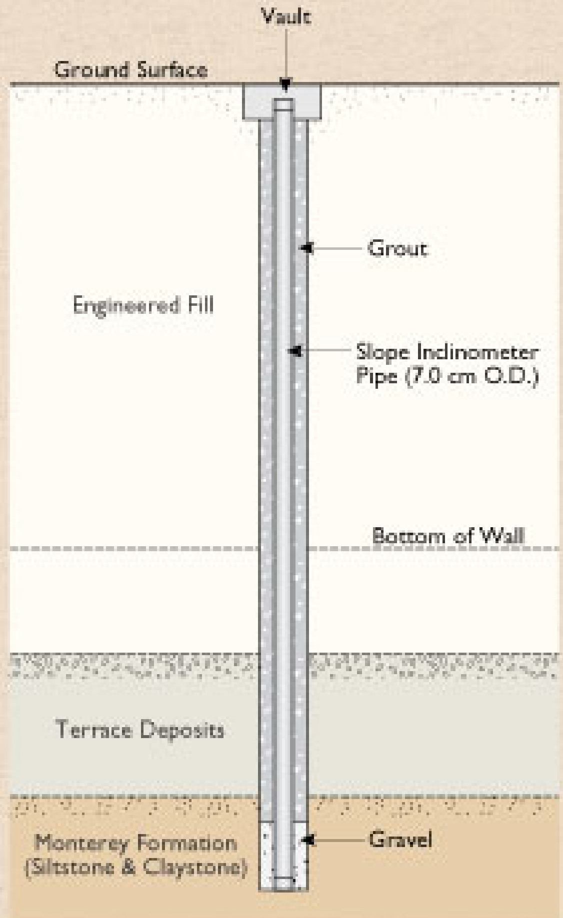

FIGURE 4:
Typical Slope Inclinometer
Installation Detail

6.2 Slope Inclinometers

To augment the survey data, three slope inclinometers were installed behind the wall. Slope inclinometer 1 was installed immediately behind the block facing at Station 5+23.32 (corresponds to the highest point of the wall as well as one of the survey arrays). Slope inclinometer 2 was installed at nearly the same stationing (Station 5+23) except that it was positioned at the back of the reinforced soil mass. A third slope inclinometer was installed at Station 6+33 immediately behind the facing block.

Slope inclinometers 1 and 2 were both installed to a depth of 28 m and socketed approximately 3 m into the Monterey formation bedrock. Slope inclinometer 3 is 17 m long and was socketed approximately 5 m into dense terrace deposits. The slope inclinometers were back-filled with a low strength cement-bentonite grout. A typical slope inclinometer installation detail is shown in Figure 4.

The inclinometers were installed on January 11 and 12, 1999 approximately 5 weeks following the completion of the wall. The first baseline reading was taken approximately 1 week after the installation date on January 18 to allow for grout settlement. Consequently, slope inclinometer readings begin roughly six weeks after the start of the survey readings. Readings have been taken on an approximate one month basis from January until December 1999.

7. INSTRUMENTATION MONITORING RESULTS

The survey readings measured horizontal and vertical movement and are shown on Figure 5 - SRW Survey Monitoring Results. Analysis of the plots show that the magnitude of movement in the horizontal direction ranged from approximately 0.5 to 2.5 cm away from the wall in the downslope direction with most values falling in the 1.0 to 2.0 cm range. Vertical movement ranged from approximately 1.5 cm to about 4.0 cm downward. Horizontal movement appeared to slow significantly after approximately 100 days following the initial reading. After the aforementioned 100 day period, the maximum additional horizontal movement was less than approximately 0.5 cm. The data further indicates that the rates of vertical movement have appeared more steady since the initial reading (rate of about 2.54 cm per 200 days). It appears that stabilization occurred approximately 250 days after the initial reading. Since the 250-day mark, vertical movement has not increased.

Measurement tolerance of the survey data was specified as plus or minus 0.6 cm. In reviewing the survey data, some slight discrepancies may indicate actual tolerance more in the range of 1.0 cm.

Recorded slope inclinometer movements are shown on Figure 6. The attached plots show movement on the “A” axis of the inclinometer. Movement was not recorded on the "B" axis. The “A”+ or positive axis is oriented towards the face of the wall. Movement was recorded over the upper approximately 15 m in Slope inclinometers 1 and 2. This corresponds roughly to the wall height of 14.9 m at that location. Movement was also recorded at slope inclinometer 3 over the upper approximately 10 m of the wall. The wall at that location is 11 m high.

The slope inclinometers indicate approximately 0.25 to 0.6 cm of movement. A majority of the movement appears to have occurred between 100 to 200 days after the base line reading (ie. Approximately 150 to 250 days following the completion of the wall). This movement is in good agreement with the survey data which indicates a similar amount of movement over the same time period. S1-1, S1-2 and possibly S1-3 indicate some deformation after the aforementioned stabilization time period. This movement is very small (ie. 0.1 to 0.3 cm) and is most likely related to strains associated with landscape water infiltration.

Slope inclinometer readings were also recorded over the time period during which the recent Hector Mine earthquake occurred. Figure 7 shows the results of the slope inclinometer readings from October 1 to October 19. Given that post construction wall movements had essentially ceased by October 1st, examination of this set of readings would record any movements associated with the Hector Mine earthquake. The slope inclinometer plots contained in Figure 7 indicate that no significant movements were incurred at the wall during the Hector Mine Earthquake.

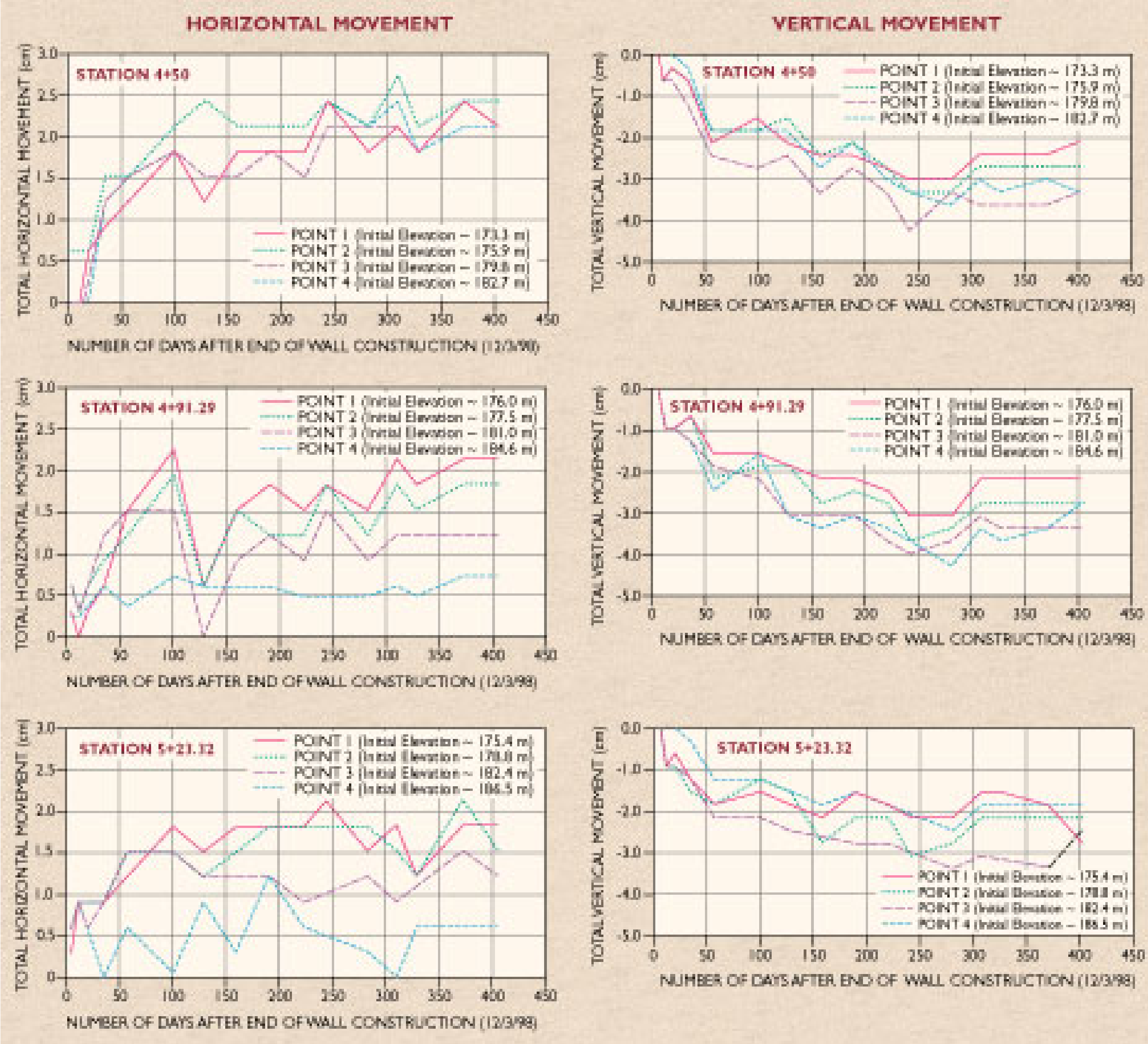

FIGURE 5: SRW Survey Monitoring Results

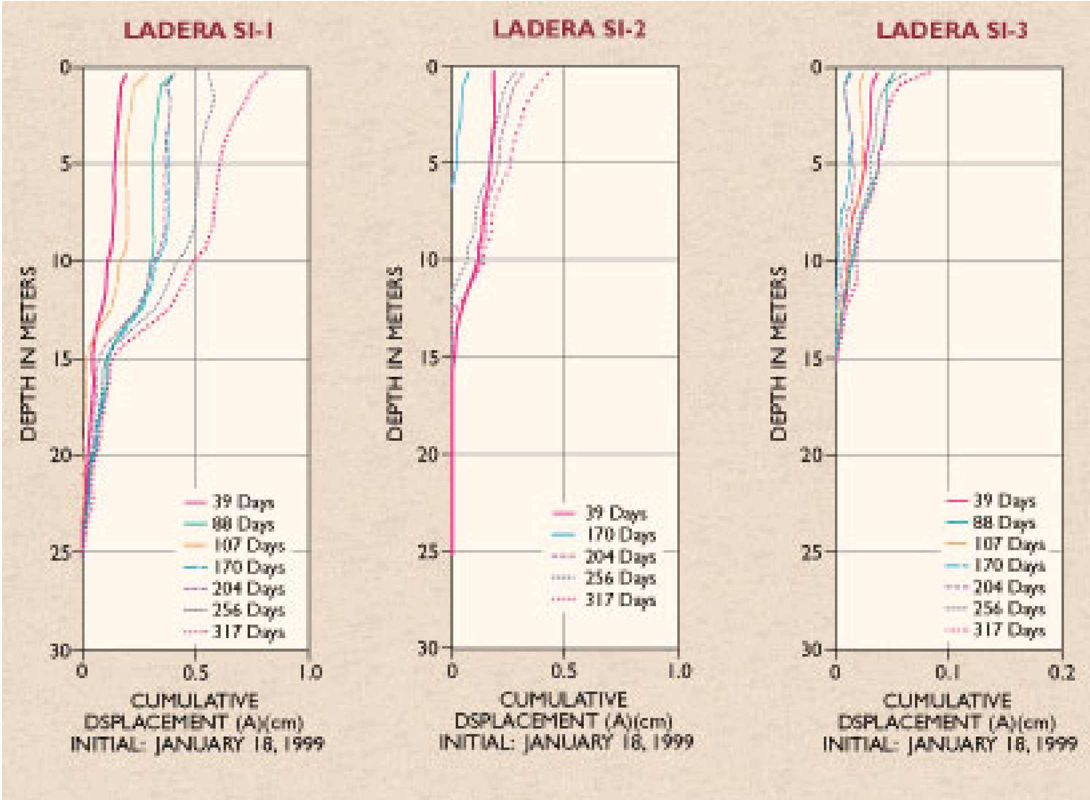

FIGURE 6: Slope Inclinometer Plots for SI-1, SI-2 and SI-3

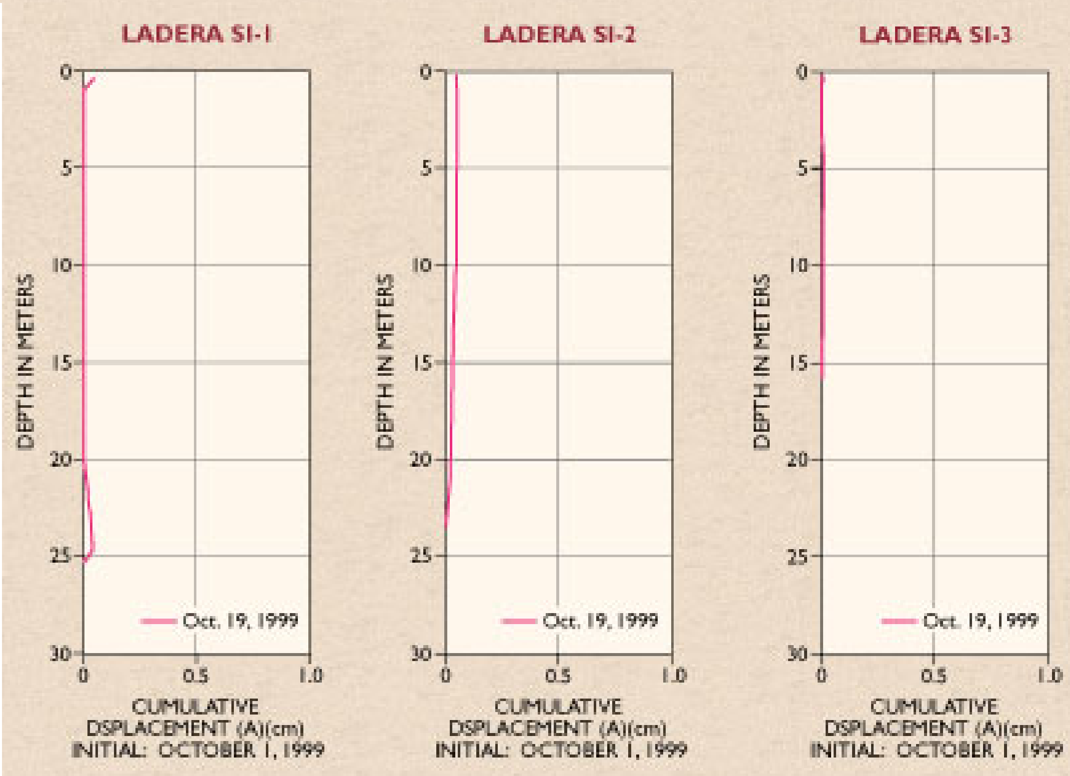

FIGURE 7: Slope Inclinometer Plots During Hector Mine Earthquake

8. DISCUSSION AND CONCLUSIONS

Total post construction horizontal and vertical movements of the wall system are on the order of 0.2% to 0.3% of the total wall height with a majority of the movement occurring over an 8 month time period. These movements were significantly smaller than expected but occurred over a longer time period than originally estimated. The small magnitudes of post construction movement are most likely related to the conservative design procedures and the fact that seismic design controlled. The relatively long time for post construction vertical and horizontal movements to stabilize is believed to be related to the rate of primary consolidation for the relatively fine grained nature of the fill soils used in the reinforced and retained zones.

Very little data exists within the literature in regards to post-construction wall movements of SRW's. Much of the literature in regards to wall movements has concentrated on movements during construction. Bathurst (1994) describes post construction movements for an under-reinforced" SRW wall instrumented as part of a FHWA research project. This wall (the Algonquin wall) was 6.1 m in height and had a 1:20 (horizontal to vertical) face batter. Recorded post construction horizontal movements were on the order of 17 mm (i.e. roughly 0.25% of the height of the wall) occurring over a 109 day or 3 month period. Post construction movements were also recorded for a 3.5 m wall constructed on the campus of the University of Wisconsin - Platteville (Wetzel, et.al., 1995). Both horizontal and vertical movements were recorded for this wall using survey targets mounted on the facing block units. Post construction horizontal movements recorded by the survey targets were approximately 0.05% of the wall height. Post-construction settlement or vertical movements were not recorded.

The recent Hector Mine Earthquake which is estimated to have produced a horizontal ground acceleration of up to 0.1g did not induce any significant movements in the wall. This result is consistent with observations recorded at various MSE walls in the Los Angeles area following the Northridge earthquake (Sandri, 1997). However, even the estimated maximum horizontal ground acceleration at the crest of the wall during the Hector Mine earthquake was roughly 1/2 of the design value. Consequently, the real test of the wall will be during a large seismic event. The slope inclinometer and survey monitoring devices are permanent and will be monitored on a periodic basis over the next several years. This will allow future measure of earthquake induced deformation, should a large earthquake strike the area.

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