Geotechnical Engineering Practice in Overconsolidated Clays, San Diego, California

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A significant portion of the San Diego metropolitan area is underlain by overconsolidated clays. The erosion of overburden soils has been the primary mechanism for overconsolidation. A decrease in regional water levels has also contributed to the relatively high levels of overconsolidation observed in several soil formations. Overconsolidated clays within the San Diego area are generally Cretaceous to Pleistocene in age and are characterized by a high expansive potential and low residual shear strength. The current status of geotechnical engineering practice in overconsolidated clays within the San Diego metropolitan area is discussed, particularly as it relates to foundation engineering and slope stability analysis.

The population of San Diego has increased tremendously since the late 1940s. A major portion of this population growth has occurred in areas underlain by overconsolidated (OC) clays. Except for the immediate coastal area, the topography of the San Diego metropolitan area generally consists of mesas and incised canyons. This topography requires extensive hillside construction techniques to provide building pads for residential and commercial developments. Construction earthwork (grading) operations involving excavation or filling, or both, in excess of 33 m (100 ft) in elevation are not uncommon. Hence, significant quantities of OC clays can be generated or exposed during the grading operations for a large residential or commercial project. The geotechnical characteristics of these OC clays has often resulted in damage to building foundations and ancillary site improvements. Deep-seated landslides and surficial slope failures, in both natural soil formations and man-made fills, have also occurred because of the presence of OC clays.

GEOLOGICAL AND GEOTECHNICAL CHARACTERISTICS

The metropolitan San Diego area is located in the Coastal Plain province encompassing the strip of land from the Pacific Ocean to approximately 8 to 16 km (5 to 10 mi) inland and parallels the coastline from the Mexican border to the greater Los Angeles area (Figure 1). Granitic rocks of the southern California batholith outcrop east of the Coastal Plain province and typically possess a shallow (less than 1.5 m) soil mantle. OC clays occur primarily within the Coastal Plain province where they are present in marine and nonmarine sedimentary deposits. Except for local faulting or folding, the inclinations of OC clay layers are nearly horizontal with regional dips on the order of 5 to 10 degrees.

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Two previous regional studies that included evaluations of the geotechnical engineering characteristics of OC clays throughout San Diego County were performed by Kennedy (*1*) and by Pinckney et al. (*2*). The study by Kennedy (*1*) consisted of detailed geologic mapping and analysis of the San Diego metropolitan area. As part of his work, numerous samples of OC clays from different soil formations were obtained and were tested for Atterberg limits, grain size, and mineralogy. Studies by Pinckney et al. (*2*) evaluated the geotechnical engineering characteristics of OC clays in areas of known or suspected landslides.

The results of laboratory tests performed by Kennedy (*1*) on OC clays over a wide area of San Diego County are summarized in Table 1. A review of Table 1 indicates that, with the exception of the bentonite layers of the Otay Formation, the Atterberg limits of the OC clays from the various soil formations are comparable, as is the clay fraction of the soil. Individual test results were not presented by Pinckney et al. (*2*); however, they reported that measured geotechnical engineering properties of OC clays from the Friars Formation exhibited similar values to those reported by Kennedy (*1*). Atterberg limit test results from the Mission Valley, Ardath shale, and Del Mar Formations were higher than those presented by Kennedy (*1*). In general, their test results indicated a liquid limit between 75 and 85, with plasticity indices ranging from 40 to 60. Residual shear strengths measured in consolidated-drained (CD) direct shear tests consisted of effective friction angles of 6 to 12 degrees, with effective cohesion values less than 9.5 kPa (200 psf). All of the laboratory test results reported by Pinckney et al. (*2*) were performed on OC clays located within known or suspected landslide areas. Additionally, the majority of their tests were performed on clays obtained within shear zones believed to represent the basal sliding surfaces of the landslides.

The Friars Formation has been the most extensively studied of the various soil formations containing OC clays because of its significant areal extent and, hence, impact on residential and commercial developments. The mineralogical composition of the Friars Formation is primarily montmorillonite, with traces of kaolinite and quartz, whereas the Atterberg limits generally range from a liquid limit of 45 to 80 with a plastic limit of 20 to 30 (*1*). In situ water contents are typically 15 to 20 percent, resulting in a liquidity index of approximately 0 to 0.10. Neither of the two referenced regional studies included consolidation tests or maximum shear strength tests on any of the OC clays. Consolidation tests performed by the author for numerous development projects in the San Diego area indicate that the overconsolidation ratio (OCR) of the Friars Formation ranges from approximately 10 to 20. It has been estimated that 60 to 120 m (200 to 400 ft) of overburden soil has been eroded above the top of the Friars Formation. Direct shear tests performed

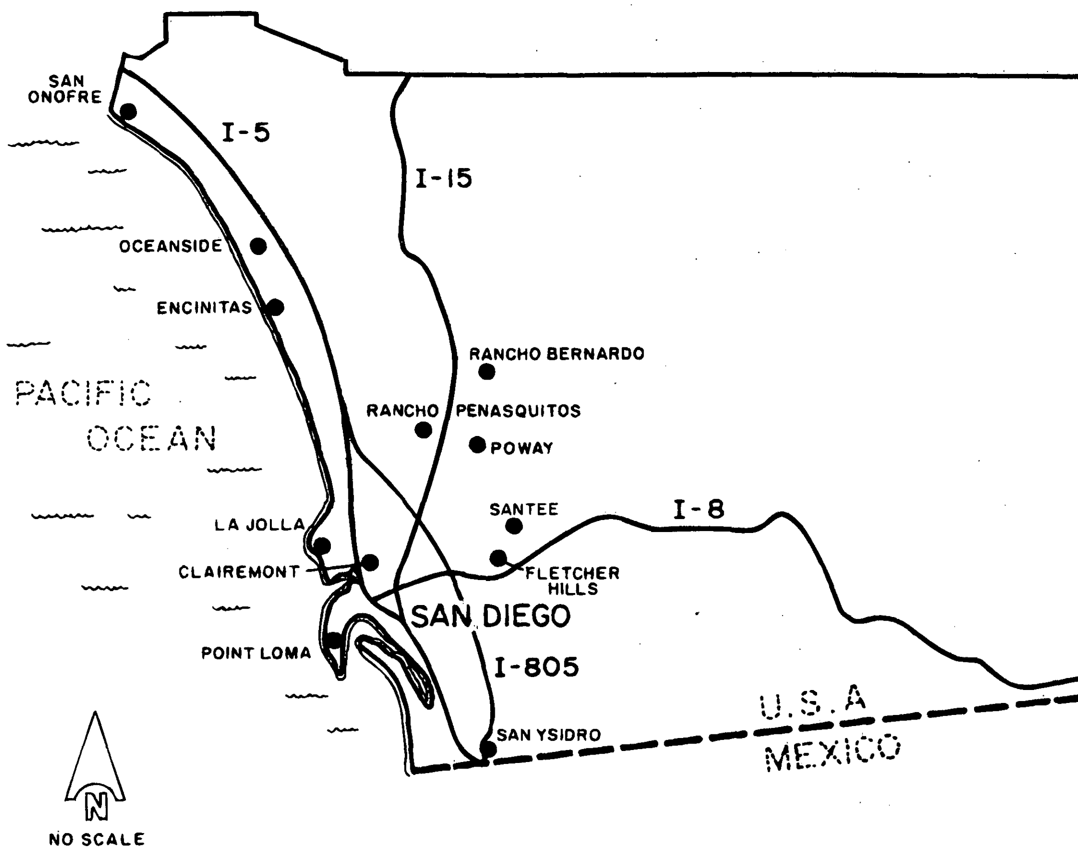

FIGURE 1 Map of San Diego County, California.

on 6-cm-diameter by 2.5-cm-thick ring samples obtained with a modified penetration sampler typically indicate an effective peak friction angle of 28 to 35 degrees and a cohesion of 14 to 35 kPa (300 to 700 psf) for "undisturbed" specimens of the Friars Formation when a single "best fit" line is applied to the direct shear test data. Triaxial testing of OC clays is rarely performed because of the difficulty in obtaining an undisturbed sample of the soil with conventional tube sampling equipment. Normalized undrained shear strength ratios for triaxial, anisotropic compression testing can be estimated from the following equation (*3*):

$$S_u/\sigma'_{vo} = 0.30OCR^{\lambda} \tag{1}$$

TABLE 1 Overconsolidated Clay Atterberg Limits and Clay Fraction

Soil Formation	Liquid Limit	Plastic Limit	Plasticity Index	Clay Fraction %<0.002mm
Del Mar	40-60	15-25	15-40	20-50
Friars	45-80	20-30	20-50	15-60
Mission Valley	40-55	10-25	15-25	20-40
Ardath Shale	40-60	20-35	20-30	10-35
Otay (Bentonite)	90-130	30-45	60-100	35-60

where

S_u = undrained shear strength,
σ_{vo} = effective overburden pressure,
OCR = overconsolidation ratio, and
λ = strength rebound parameter.

Using an average strength rebound parameter of 0.8 (*3*) and OCR values from 10 to 20 results in normalized shear strength ratios of approximately 1.8 to 3.2. Hence, for depths of interest in typical geotechnical engineering analyses (3 to 20 m), the maximum undrained shear strength would vary from approximately 190 kPa (4000 psf) at 3 m to 625 kPa (13,000 psf) at 20 m.

Residual shear strength testing is commonly performed on OC clays in areas where ancient landslides are suspected or in areas where new construction will result in excavation slopes that will expose OC clays. Current practice is to perform CD residual shear tests in a direct shear box apparatus. Ring shear testing is also becoming more common. Residual shear strength test results for the majority of OC clays typically ranges from 6 to 10 degrees (effective friction angle), with nominal values of cohesion (2.5 to 5 kPa). As discussed by Skempton (*4*), the residual shear strength friction angle is nonlinear, particularly at low levels of effective normal stress. However, the current standard of practice typically consists of using a single value of effective friction angle and cohesion for geotechnical engineering analysis.

The expansion potential of an OC clay is typically measured in a one-dimensional, free swell test known as the Expansion Index (EI) test. Uniform Building Code Standard 29-2 presents recommended test procedures (*5*). The soil specimen is remolded to a water content corresponding to approximately 50 percent saturation. It should be noted that a standard soil dry density is not specified and that a specific gravity of 2.7 is assumed for the soil. A surcharge pressure of 7.2 kPa (150 psf) is applied to the top of the specimen, and the specimen is then inundated with distilled water. The vertical strain is measured, and at equilibrium (typically 6 to 24 hr), the EI is calculated as $\Delta H/H * 1000$, where ΔH is the vertical movement (swell), and H is the original sample thickness. Expansive soil classifications based on EI test results have been developed by the Uniform Building Code as follows:

Expansion Index (EI)	*Expansion Potential*
0–20	Very low
21–50	Low
51–90	Medium
91–130	High
Above 130	Very high

EI values for OC clays are commonly in excess of 90, with several bentonite-rich OC clays exhibiting EI values above 150. The EI test is performed on soil that is finer than the No. 4 (6.35-mm) sieve. Many expansive OC clay soils in the San Diego area consist of gravel to cobble materials within a clay matrix. Houston and Vann (*6*) presented data that indicated that the expansion characteristics of a clay soil are significantly influenced by the percentage of material larger than the No. 4 sieve. Therefore, the main purpose of the EI test is to provide a qualitative assessment of the swelling potential of a soil as opposed to a parameter that can be used in the direct design of a foundation to mitigate the potential for swell of an expansive clay.

SLOPE STABILITY

Slope stability problems associated with OC clays can be divided into two categories. The first category consists of large slope movements or landslides that occur in natural soil formations along OC clay seams or lenses. These landslides may result from natural processes, such as erosion or hillside creep, or as a consequence of construction activities. Typically the sliding surface is located along a previously sheared, slickensided clay seam or lens that is at or slightly above its residual shear strength. The second category includes surficial slope movements associated with the use of OC clays for constructing fill slopes.

There are many ancient landslides within OC clays in San Diego County. A significant number of ancient landslides have been identified by Hart (*7*), Kennedy (*1*), and Pinckney et al. (*2*) in regional geologic studies. Major landslide areas include the Poway, Rancho Bernardo, Fletcher Hills, and San Ysidro areas (Figure 1). The predominant soil formations in these areas are the Friars and Otay formations. These landslides commonly have a basal surface along a low residual shear strength OC clay seam; the thickness of a clay seam can be as thin as 6 mm (0.25 in.). Ancient landslides are believed to have occurred when the climatic conditions in southern California were much wetter than today. The combination of lower effective stresses, higher soil unit weights, and active stream erosion strained the OC clays to residual shear strength values, resulting in downhill movements. Landslides then resulted because of the low residual shear strength of the OC clays. Deep-seated slope movements along OC clay seams or lenses have also occurred as a result of construction processes. The excavation at the base of a natural hillside containing an OC clay seam dipping out of slope (toward the excavation) has resulted in slope movements of varying magnitudes.

Identification and analysis of OC clay seams or lenses is a crucial part of any geotechnical investigation for a site that has, or will have, any significant topographic relief. Additionally, the presence of ancient landslides, which may have resulted in the straining of OC clay seams or lenses to residual shear strengths, is also important to identify. Because of erosion, the topographic expression of an ancient landslide is usually subdued and is not always easily identifiable from the ground surface. Stereoscopic aerial photographs can provide assistance in evaluating topographic remnants of ancient landslides, such as hummocky terrain, scarps, or altered drainages. Areas of suspected ancient landsliding, as well as areas where cut slope excavations are proposed, are typically investigated with down-hole geologic mapping techniques. A bucket-auger drill rig is used to advance a 60- to 120-cm (24- to 48-in.) diameter boring into the ground. Minimum depth of excavation is typically 3 m (10 ft) below the base of a proposed cut slope or below the basal surface of a suspected landslide. The boring is then inspected in situ by a geologist or engineer with expertise in identifying landslide materials. Geologic features used to identify landslides include the continuity and integrity of the soil materials, joints or other discontinuities, and the presence of sheared clay seams or lenses. If present, the depth and inclination of any clay seams or landslide basal surfaces are also determined for subsequent geotechnical engineering analyses. Samples of undisturbed and sheared clay materials are obtained for laboratory testing. For large projects a minimum of three bucket-auger borings are advanced to determine the continuity and three-dimensional characteristics (strike and dip) of the clay seams or lenses.

If an ancient landslide is identified within the area proposed for development or if an OC clay layer is present that may potentially affect the stability of proposed excavation slopes, laboratory testing and slope stability analyses are performed to evaluate the proposed site-grading configuration. A cross section showing a typical grading configuration and existing site condition is presented in Figure 2. Direct shear or ring shear tests are performed to determine the maximum and residual shear strengths of the OC clay material. Slope stability analyses are typically performed using commercially available computer programs. Local building code ordinances and the standard of practice result in the requirement of slope mitigation measures if the minimum factor of safety determined from the analysis is less than 1.5.

Numerous slope stabilization techniques have been used over the years to increase the factor of safety against slope movements. The most common and widely used procedure has been the construction of buttresses at the toe of the landslide or proposed slope (Figure 2). The buttress is constructed of compacted fill soil and is intended to provide an area (per unit width) of higher shear strength material, which will result in an increased resistance to slope movement. Buttress thicknesses (into the slope) can range from approximately 3 m (10 ft) (minimum width of construction equipment) to 33 m (100 ft) or more. Buttress construction begins by excavating a temporary slope into the natural hillside, which will provide the required buttress width. Because of slope stability considerations, the buttress construction is typically performed in slots parallel to the slope, with the width of a slot usually on the order of 30 to 60 m (100 to 200 ft). Using this procedure, the grading contractor builds the buttress along the length of the slope requiring mitigation. For smaller, more local areas of potential slope instability, other techniques that have been

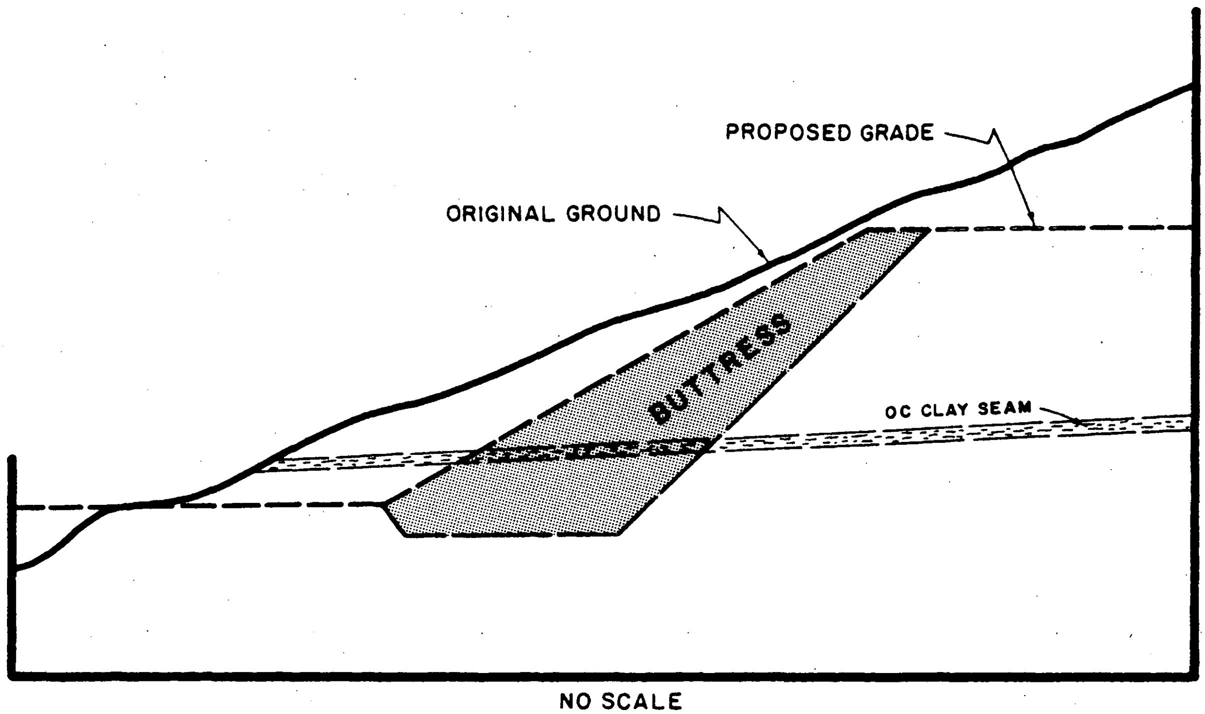

FIGURE 2 Typical cross section with buttress constructed to stabilize excavated slope.

used successfully include soil reinforced with metal or plastic strips (Reinforced Earth, geogrids, etc.) and drilled piers or caissons.

Another slope stability problem with OC clays exposed in excavated slopes is the presence of fractured or blocky clay materials. Slopes are generally excavated at inclinations of 26 to 33 degrees from horizontal and typically possess factors of safety against slope failure in excess of 1.5 (if no low-strength clay seams are present). However, the local presence of fractured or blocky clay materials in an excavation slope face can result in surficial slope instability. Where present, this potentially unstable soil is excavated and replaced with a stability fill. A stability fill consists of compacted fill soil placed within the outer 3 m (10 ft) (approximate width of earthwork construction equipment) of a slope for a height and length that encompasses the zone of fractured or blocky clay material. Other forms of mitigation, such as retention of fractured materials with wire or jute netting, can be used when aesthetics and landscaping are not of concern.

Slope stability problems can also result from the use of OC clays as fill soils when highway or railway embankments or slopes for development projects are constructed. Stability problems are typically more of a surficial nature with soil sloughage or "pop-outs" occurring in fill slopes to depths of 0.6 to 1.2 m (2 to 4 ft) below the surface. The areal extent of instability is typically on the order of 15 to 60 m (50 to 200 ft) in length (parallel to slope) and can include the entire height of the slope face. Postconstruction saturation of the outer portion of the fill slope is the primary reason for these movements. The soil to a depth of 0.6 to 1.2 m (2 to 4 ft) below the slope face can become saturated because of rainfall or irrigation waters. Saturation results in a decrease in effective stresses, which reduces the shear resistance of the soil. The soil unit weight also increases because of saturation. Additionally, insufficient compaction of the outer 1 to 1.2 m (3 to 4 ft) of a fill slope inclined at 26 to 30 degrees can result in lower shear strengths than those used for the slope stability design. Typical design strategies consist of not placing clay materials within a minimum horizontal distance of 3 m (10 ft) from the slope face to reduce the potential for surficial slope instabilities. However, the large volume of clay soil excavated from other areas of the property or a limited area available for disposing of clays generated during site excavation operations can result in clays soils being placed within the outer portions of a fill slope. In these cases, using drought-resistant vegetation with deep roots and controlling the volume of irrigation water are the typical techniques used to reduce the potential for surficial slope instability. The compactive effort applied to the outer portion of the fill slope can also be increased by overbuilding the slope (typically 1.5 to 3 m) and then trimming the soil back to the final fill slope design grade.

FOUNDATION ENGINEERING

The relatively high expansive potential of OC clays can present problems for residential and commercial buildings as well as other on-grade improvements. Expansive soils can be a problem in areas that are excavated to finish grade and that expose OC clays or in fill areas where the compacted fill consists of OC clay. Various methods of expansive soil mitigation have been used, including removal and replacement, recompaction, and the use of structurally designed posttensioned foundations and slabs-on-grade.

Geotechnical investigations for residential and commercial developments in areas of suspected expansive OC clays usually include soil borings or backhoe trenches to determine the areal extent and depth of expansive soils. Samples of the soils that may affect foundation design are obtained for laboratory testing. Based on the laboratory test results (primarily EI and Atterberg limits) and project economics, several expansive soil mitigation procedures may be viable.

Removal of expansive soils exposed at building finish pad grade and replacement with non- to low-expansive soils (EI < 50) is a common technique. This technique is often economically feasible when the export of expansive soils and replacement with non- to low-expansive soils can be performed within the boundaries of the project. Typical projects for which this would apply include large residential and commercial developments in which the expansive soils can be "wasted" in nonstructural areas, such as parks and landscape areas. Additionally, a source of non- to low-expansive soils (i.e., sand) within the materials being excavated for general site grading will enable this technique to be economically viable. Removal and replacement of expansive soil are generally not economical for small project areas or single building construction projects. The depth of removal (undercut) typically extends 1 to 1.5 m (3 to 5 ft) below finish pad grade. Chen (*8*) recommend using a minimum thickness of 1.2 m (4 ft) of non- to low-expansive soils when expansive soils are present at finish grade. The depth of expansive soil undercut is based on the expansive potential of the soil (as defined by the EI), structural footing loads and floor loads, and composition of areas immediately surrounding the building (e.g., impervious cover, such as pavement or hardscape versus permeable cover, such as landscaping). Removal of expansive soil is typically performed to a horizontal distance of 1.5 to 3 m (5 to 10 ft) beyond the perimeter of the structure or affected improvements. This technique can also be used when expansive soils are used for fill soils. The upper 1 to 1.5 m (3 to 5 ft) of a building pad can be "capped" with non- to low-expansive fill soil.

A problem that can result from the above procedure is the so-called "bathtub" effect, which occurs when water infiltrates down through the permeable non- to low-expansive soil and accumulates at the surface of the underlying expansive clay soil. Continued infiltration can result in a groundwater mound beneath the structure. Water vapor can then migrate through concrete slabs-on-grade or hardwood floors and saturate overlying carpet or other floor coverings. The potential for moisture problems can be reduced if a drain system is installed beneath the structure or if the surface of the expansive clay soil is graded so that the infiltrated water drains away from the structure.

Recompaction of expansive soils at higher water contents or lower dry densities is sometimes performed when the removal and replacement of the expansive soils is cost prohibitive. This procedure consists of excavating the soil to a depth of approximately 1.3 to 1.5 m (4 to 5 ft) below finish grade and (*a*) increasing the moisture content to approximately 4 to 6 percent above the optimum moisture content and/or (*b*) recompacting the soil at a lower relative compaction (e.g., 85 percent relative compaction in lieu of 90 percent relative compaction). As would be expected, the compaction of clayey soil at moisture contents 4 to 6 percent above optimum (approximately 90 percent saturation) can be extremely difficult. In addition, most municipal building codes within the county (including the city of San Diego) require a minimum relative compaction of 90 percent (ASTMD-1557-90) for fill soils that will support building foundations or other structural improvements. Although one- and two-story commercial and residential buildings typically have nominal foundation loads [less than 95 kPa (2000 psf)], it can be very difficult to have a city or county official agree to waive the 90 percent compaction requirements to allow a lower relative compaction to reduce the expansion potential of the soil.

A third option for designing foundations in expansive soil is to use a structurally designed footing and slab system. This will typically consist of a posttensioned slab with deepened exterior footings and possibly interior grade beams. Design of the foundation is performed by a structural engineer based on soil parameters provided by the geotechnical engineer. A majority of this design has also been codified by governing agencies. Other techniques successfully used in other parts of the country for expansive soil mitigation, such as drilled piers, have not been used in the San Diego area.

Typical minimum foundation design recommendations for residential and light industrial/commercial structures based on the EI of the soil within 1 to 1.5 m (3 to 5 ft) of finish pad grade are presented in Table 2. These recommendations are based only on the expansion characteristics of the underlying soils and do not consider the bearing capacity or settlement potential of the soil.

Bearing capacity of shallow foundations for residential and light industrial/commercial structures is rarely a problem within OC clays. Allowable foundation bearing capacities of 95 to 288 kPa (2000 to 6000 psf) with foundation dimensions on the order of 30 to 60 cm (12 to 24 in.) in depth and 45 to 122 cm (18 to 48 in.) in width are fairly typical. Deep foundations supporting heavy structures (or light structures overlying compressible soils) typically use drilled piers or driven piles embedded in the underlying OC clay. Drilled piers are commonly 61 to 122 cm (24 to 48 in.) in diameter, and driven piles commonly consist of 30- to 61-cm (12- to 24-in.), square precast, prestressed concrete piles. Steel H-piles or pipe piles are occasionally used also. The predominant deep foundation used for heavy structures is a drilled pier excavated without casing or slurry support. Steel casing and/or bentonite slurry is used in areas where there is groundwater or unstable soil conditions. The capacity of deep foundations is determined based on the side resistance and end bearing resistance of the OC clay. The current standard of practice is to design deep foundations on the basis of the undrained shear strength (S_u) of the OC clays (i.e., a total stress analysis). Side resistance of the pier or pile is calculated using the undrained shear strength of the clay and an adhesion factor (α). The adhesion factor for stiff, overconsolidated clays is typically in the range of 0.4 to 0.6 (*9*). Undrained shear strength values along the length of the pier or pile can be divided into layers or averaged together as a single value. End bearing capacity is typically determined from the total stress deep foundation bearing capacity formula $q = S_u * N_c$ where $N_c = 9.3$ and q = ultimate bearing capacity. Factors of safety applied to the side resistance and end bearing resistance typically range from 1.5 to 3.0. Very few documented results of load tests performed on drilled piers or driven piles embedded in OC clays exist for the San Diego area. The results of a drilled pier load test in the Ardath Shale Formation designed to measure side resistance only were reported by Spang (*10*). The actual drilled pier capacity was much higher than that predicted, with commonly used undrained shear strength values [190 to 380 kPa (4000 to 8000 psf)]. This is believed to be the result of conservative undrained shear strength values or the use of total stress analysis in lieu of the more fundamentally correct effective stress analysis or the result of both.

CONCLUSION

Geotechnical engineering in the metropolitan San Diego area is greatly influenced by the presence of OC clays. The primary geotechnical engineering challenges associated with these clays are slope stability and foundation design. Laboratory testing procedures have been developed to assist characterizing the expansive potential and residual shear strength of OC clays. Numerous construction and design techniques have been used to reduce the potential for detri-

TABLE 2 Typical Foundation Recommendations on Expansive Soils (Residential and Light Industrial and Commercial Structures)

	Foundations		Concrete Slab-on-grade	
Expansion Index	Depth below finish pad grade (cm)	Steel Reinforcement	Thickness (cm)	Steel Reinforcement
0-50	30	2 - #4 bars, 1 top & 1 bottom	10	6x6-10/10 welded wire mesh or #3 bars @ 61 cm
51-90	46	4 - # 4 bars, 2 top & 2 bottom	10	6x6-10/10 wwm or #3 bars @ 46 cm
91-130	61	4 - #4 bars, 2 top & 2 bottom	10 to 12.5	#3 bars at 46 cm
130+	61 to 91	4 - #5 bars, 2 top & 2 bottom	10 to 15	#4 bars @ 38 cm

Note: Remedial grading operations and/or structurally designed post-tensioned foundation systems are also utilized for soils having an Expansion Index above 130.

mental movements of OC clays; however, no method can completely mitigate the movements that can be generated by expansive soils. Continuing research should provide assistance in assessing the geotechnical properties of OC clays and provide additional methodologies to mitigate the presence of OC clays on slopes and foundations.

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