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Discussion of "San Jacinto Monument Case History" by Jean-Louis Briaud, Jennifer Nicks, Keunyoung Rhee, and Gregory Stieben

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The authors are owed much credit for making this interesting case history available to the profession. The historic narrative and, in particular, the account of Professor Dawson's' approach to the settlement prediction at the dawn of the geotechnical profession adds an extra dimension to the paper. Indeed, case records, in particular those from a long period of observations, such as the subject case, are always exciting and well worth perusing in detail. The discussers appreciate the authors' evaluation and conclusions, but the unusually long observation time and the clear and open presentation of the data, do encourage pursuing a differing perspective. In so doing, we do not intend to suggest that the authors' treatment of the case records would be wrong. However, we believe that a somewhat different evaluation would emerge on considering a few conditions not included by the authors.

The authors have fitted the settlement records to a constant elastic modulus value within an influence depth taken as twice the monument width (38 m) and have excluded the general subsidence from the calculation. However, an elastic modulus approach applies a linear response of the soil to a stress increase, disregarding both the fact that the response of a compressible soil is neither linear nor independent of the ratio of the stress increase to the existing stress. When consolidation test results are available, they provide a more representative reference to settlement calculations. The authors do state that also the consolidation parameters produced by Professor Dawson's students were used in the back calculation of the settlement records. However, while the distributions of the compression indices are included in the paper, the associated void ratio values, used by the authors, are not. Fig. 1 shows a replot of the authors' virgin and reloading compression indices, C_c and C_r, obtained from the consolidation tests. The values are somewhat scattered and the ratio between the indices is much smaller than one would expect from similar tests on the Beaumont and Montgomery formations, as presented in numerous other studies, e.g., Focht et al. (1978), Mahar and O'Neill (1983), Williams (1987), Endley et al. (1996), and Javed (2005). The results presented by Endley et al. (1996) and Javed (2005) from a large number of consolidation tests are shown in Figs. 2 and 3.

The ratio between the Cc and Cr shown in Figs. 2 and 3 is about 6, whereas the ratio of the authors' indices (Fig. 1) is about half that. However, just comparing compression indices do not indicate the true ratio between virgin and re-loading compressibility, as the compressibility is an expression of both the index and the void ratio. The need for expressing the compressibility in two separate values is avoided by employing the Janbu modulus number approach. The modulus number (m or m_r) is a direct function of compression index (C_c or C_r) and void ratio (e_0) (Janbu 1963, 1998; CFEM 1992; Fellenius 2006), as demonstrated in Eq. 1.

$$m = \ln 10 \frac{1+e_0}{C_c} = 2.3 \frac{1+e_0}{C_c} \qquad (1)$$

Because Endley et al. (1996) and Javed (2005) also report the void ratio values associated with the compression indices, the pairs of compression index and void ratio values can be converted to modulus numbers as shown in Figs. 4 and 5, indicating a ratio between re-loading and virgin compressibility of about 8 for the tests.

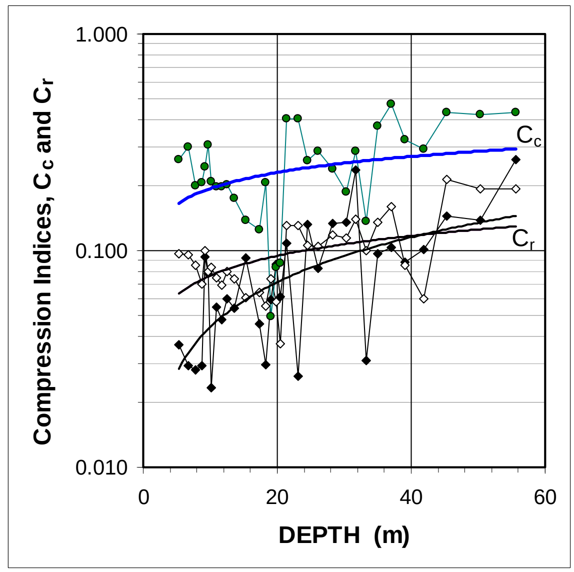

Fig. 1 The authors' compression indices in a common diagram

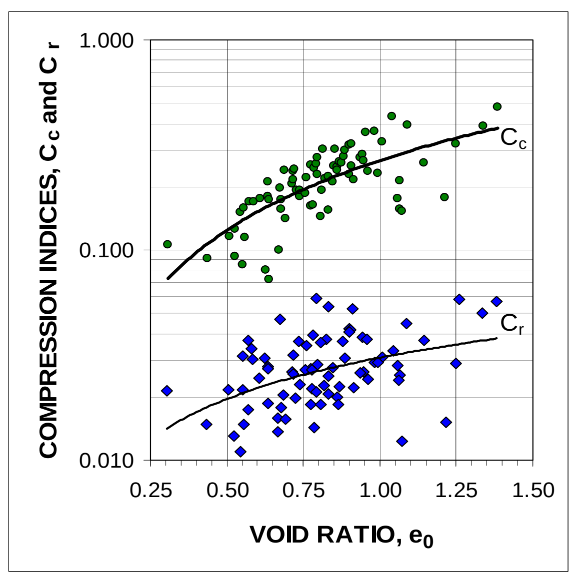

Fig. 2 Compression indices versus void ratio for Beaumont clay (adapted from Endley et al. 1996)

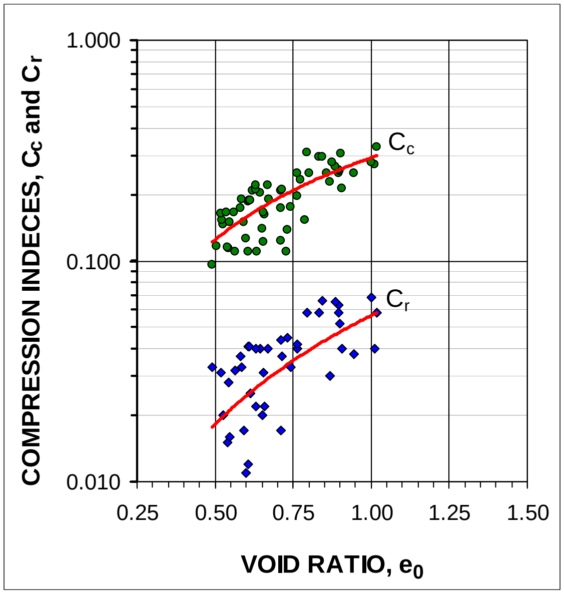

Fig. 3 Compression indices versus void ratio for Beaumont clay (adapted from Javed 2005)

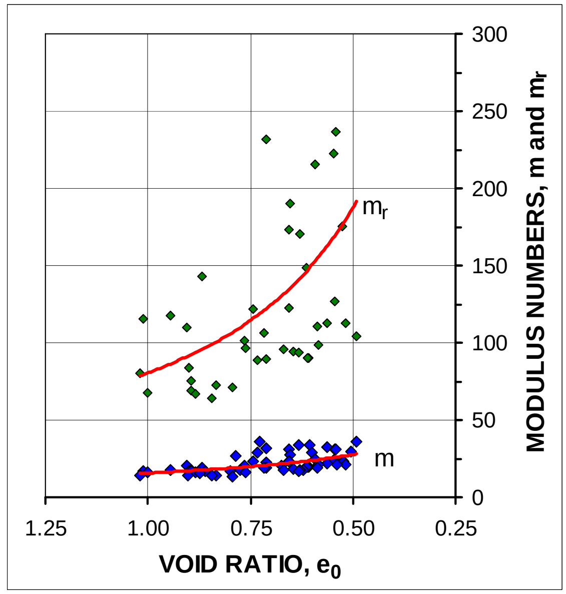

Fig. 5 Modulus numbers m and m_r versus void ratio for Beaumont clay (adapted from Javed 2005)

The Beaumont and Montgomery clays are overconsolidated, as the authors also indicate. The preconsolidation is largest near the ground surface and diminishes with depth. For example, Mahar and O'Neill (1983) indicate that the Overconsolidation Ratio, OCR, from consolidation tests is about 3 at 6 m depth decreasing to about 2 at 20 m depth. Williams (1988) indicates a variation of OCR from 15 to 4 over the same depths. Williams (1987) indicates that the preconsolidation margin in the same soils is 1,000 KPa to 2,000 KPa at 6 m depth reducing to about 1,000 KPa at 20 m depth. The margins correspond to OCR values of about 20 and 4, respectively. Because the source of the preconsolidation is desiccation in the distant past, it is probable that the preconsolidation margin continues to reduce below 20 m depth. It is expected, however, that some preconsolidation is present also at large depth.

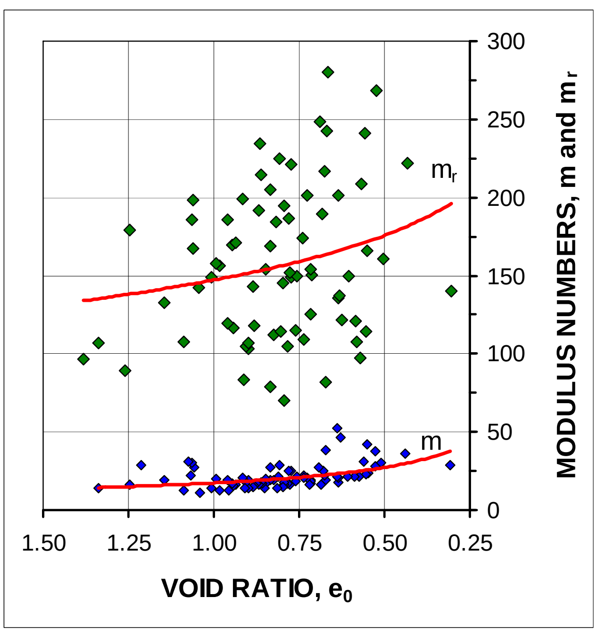

Fig. 4 Modulus numbers m and m_r versus void ratio for Beaumont clay (adapted from Endley et al. 1996)

The authors plotted the settlement records in a linear time scale. The discussers have replotted the settlement records in Fig. 6 using both linear and logarithmic time scales in the same diagram. The logarithmic plot suggests that after 1940, rather than continuing to reduce with log-time, the rate of settlement increased, reaching a final value of 328 mm. Speculatively, the trend from the first few years after 1936 is extrapolated (the dashed line), implying a just short of 100 mm total consolidation differential settlement between the bench mark and the monument from the structure and fill alone. This is, possibly, the settlement for the case of no pumping, i.e., no reduction of pore pressure due to mining of ground water. We would indeed expect that Professor Dawson's 1936 prediction of 181 mm was intentionally conservative, as suggested by our speculative extrapolation. In 1936, the full consequence of the well pumping in the area was not known and the large regional settlement was not anticipated. Professor Dawson's prediction would likely not have considered additional consolidation from the mining of ground water in deep wells several kilometers away.

The authors report that the pumping in the deep aquifers in the general area has resulted in the ground surface in the vicinity of the monument settling more than 2 m since the construction of the monument in 1936. The authors do not include information on the distribution of the pore pressures with depth. It appears that the authors' back-calculations have been made assuming hydrostatic pore pressure distribution. However, the US Coast and Geodetic Survey has measured drawdown pressures equal to negative heads of several hundred feet at large depth (Delflache 1978). Mining of water by pumping in deep wells started in the late 1920s and early 1930s and became extensive toward the end of the Second World War through the 1960s (Garcia 1991). In the mid and late 1970s, the State of Texas and affected counties began active effort to reduce the pumping and revert to use of surface water. The objective was to halt the subsidence by decreasing the rate of mining the deep ground water aquifers thereby ending the 40-year trend of incessant lowering of pore pressures, possibly even reversing the pore pressure reduction (Gabrysch and Bonnet 1975, Kasmarek and Robinson 2004, and Barbie et al. 2005). The similar situation in the Bangkok delta, Thailand, has shown cessation of pumping to be to be successful in reversing the trend of draw down and stopping ongoing subsidence (Fox et al. 2004; Seah 2006).

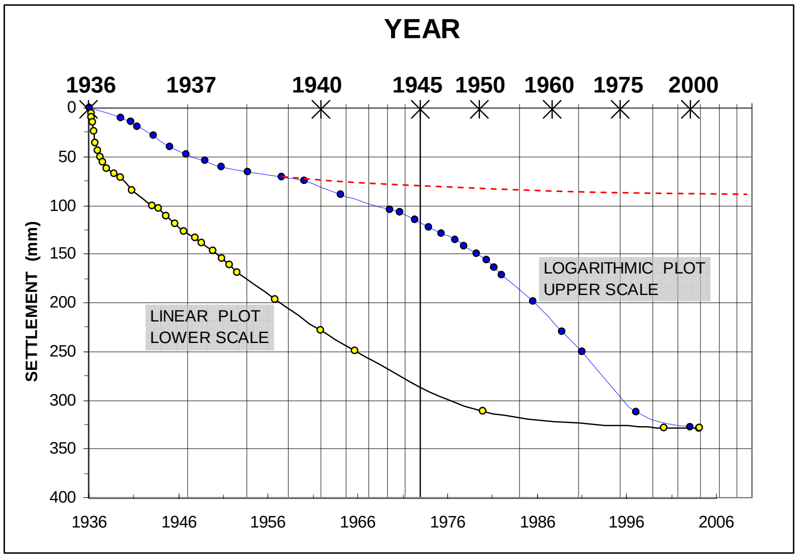

Fig. 6 Average monument settlement versus years in linear and logarithmic scales

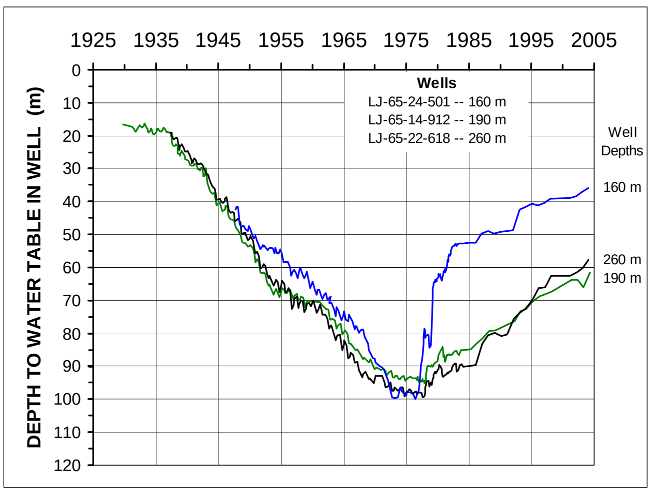

Fig. 7 Records of depth to water table from 1929 through 2005 in three nearby wells (adapted from Barbie et al., 2005).

Barbie et al. (2005) reported depths to water table measured in three deep wells located about 8 Km to 46 Km away from the monument site. The records are presented in Fig. 7 The three curves indicate that the deep well pumping started to cause a draw down in the general area in about 1938, and that the rate of draw down increased from about 1943 onward. By 1975, the draw down was close to 100 m head of water of which about half to two-thirds was recovered between 1975 and 2005.

In Fig. 8, we have superimposed the records of changing depth-to-water-table on the settlement development plotted in log-time scale. Comparing the two sets of records indicates that the rate of settlement and rate of draw down show the same trend, and in about 1975, when the draw down trend was reversed, the settlement rate reduced, indicating near-end of consolidation.

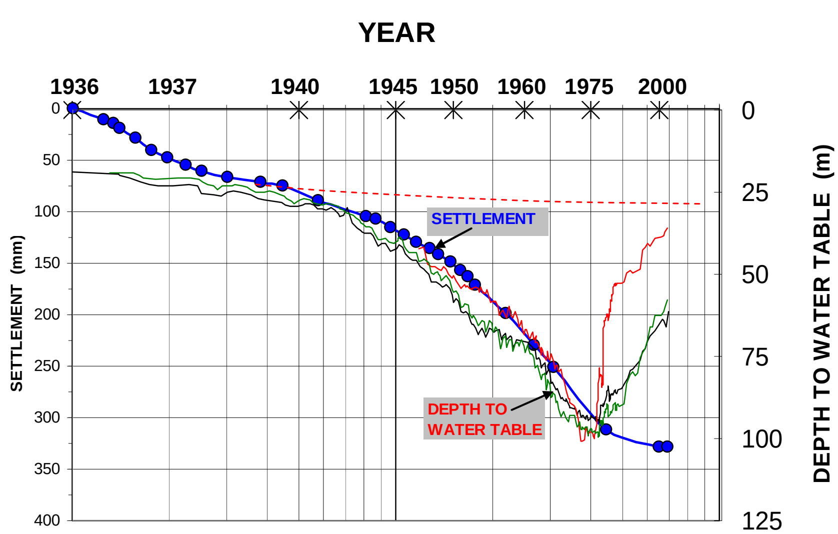

Fig. 8 Records of depth to water table superimposed on the settlement records — Log-time scale

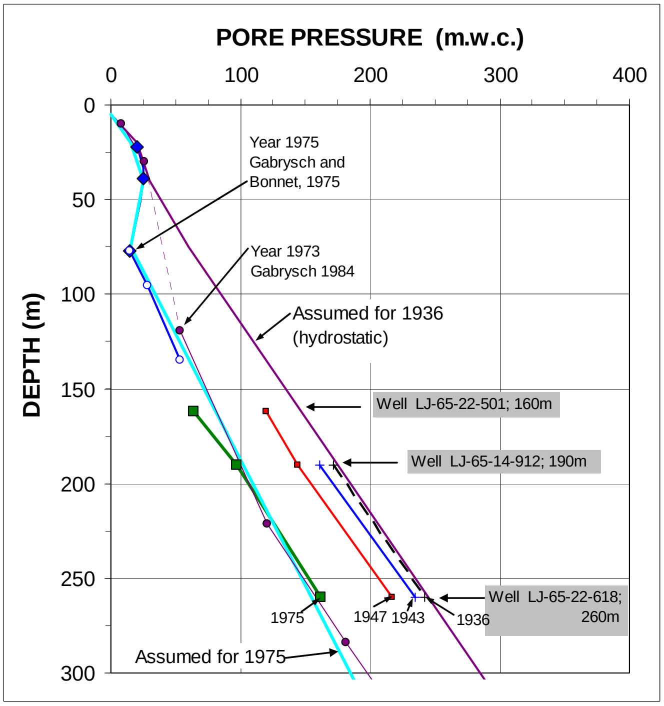

Fig. 9 Measured distribution of pore pressures in 1936, 1943, 1947, and 1975 ("m.w.c." = metre water column).

The three wells are installed to different depths and the depth to the water table in each well can be used to indicate a pore pressure distribution. Fig. 9 shows distributions in 1936, 1943, 1947, and 1975. Also shown are records of pore pressure measured in 1975 and 1973 reported by Gabrysch and Bonnet (1975) and Gabrysch (1984), respectively from eight well sites installed in Pasadena about 20 Km west of the monument location. The distributions indicate that below a depth of about 75 m, the pore pressure distribution is approximately linear with a downward gradient, ***i***, of 0.75.

Using the UniSettle software (Fellenius and Goudreault 1996), we first calculated the stress distributions of the four separate entities of imposed stress change (Fig. 10) from the monument, the two fills, and the excavation between 1936 and 1975. The loads are placed at the bottom of the excavation and the stresses are distributed down and sideways into the soil body according to Boussinesq method.

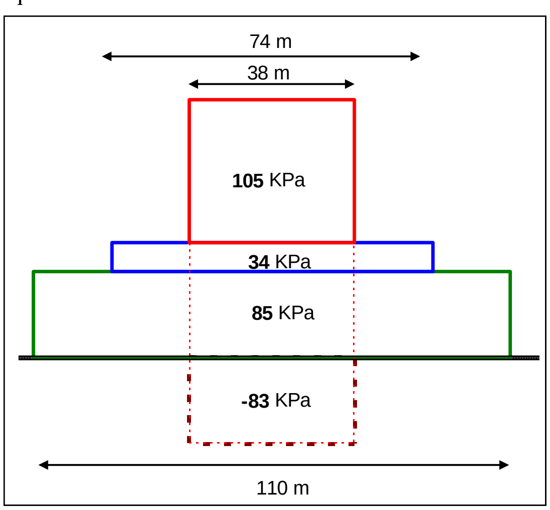

Fig. 10 Input of excavation unloading "stress" and loading stress values of the monument and fills, and side lengths of square areas affected.

The calculated stress distributions from the ground surface to the somewhat arbitrarily chosen depth of 275 m for the monument location and for the benchmark location are shown in Fig. 11. The distributions show that the imposed loads will have a small effect also at the 275 m depth at the benchmark location 45 m outside the fill area footprint. In terms of the monument width (B = 38 m), the depth corresponds to an influence depth of almost 8 B, considerably larger than the 2 B mentioned by the authors. In terms of the 110 m width of the fill footprint, the depth corresponds to almost 3 B.

Three separate settlement calculations are performed employing the mentioned stress distributions. The soil profile is very simplified and input as 30 m of clay with density of 1,800 kg/m^3, followed by 270 m of clay with a density of 2,000 kg/m^3 on non-compressible soil at 300 m depth. In the first set of calculations, we matched the 100 mm total differential settlement between the monument and the benchmark for constant pore pressure. Assuming that the preconsolidation margin is larger than the imposed stress change, we found that the re-loading modulus number, m_r, had to be 200, which value is within the range of the values indicated in Figs. 4 and 5. The same input calculates the settlement for the benchmark location 45 m outside the edge of the fill area to 6 mm, i.e. the bench-mark is practically uninfluenced by the monument and fills.

In the second set of calculations, we matched the 328-mm measured differential settlement, which resulted in a required re-loading modulus number of 55. (The calculated settlement for the benchmark was 20 mm). While a re-loading modulus number of 200 can be considered within the realm of actual values for the soil, a re-loading modulus number of 55 is far below what the reasonably smallest re-loading modulus number would be. Clearly, to match the 328 mm value, the imposed stress must be acting also in the virgin range of the soil compressibility, which conflicts with the fact that the preconsolidation margin is larger than the imposed stress. Therefore, an acceptable match to the 328 mm measured settlement cannot be achieved unless also the 2.2 m general subsidence due to the reduced pore pressures is included in matching the calculations to the measured differential settlement between the monument and the benchmark.

For the third set of calculations, therefore, a match to the 2.2 m overall settlement and the 328 mm differential settlement was obtained with input of a virgin modulus and re-loading modulus numbers of 25 and 200, respectively, which, according to the data shown in Figs. 4 and 5, are a representative values for the Beaumont and Montgomery clays. (For persons unfamiliar the with the Janbu approach and needing a reference to values of C_c-e_0, for an assumed void ratio of 0.5, the mentioned modulus numbers convert to C_c and C_r values of 0.14 and 0.017, respectively). We also assigned a preconsolidation margin to the soil ranging from 1,000 KPa near the ground surface and reducing —somewhat arbitrarily — to 200 KPa at 300 m depth. The initial and final pore pressure distributions were input as indicated in Fig. 9 for 1936 and 1975, respectively. This made the loads imposed from the monument and the fills to act on the re-loading condition, and the pore pressure reduction to act on mostly virgin conditions below about 50 m depth. The input fitted the calculations to the measured monument and benchmark differential settlements as well as the 2.2 m total settlement. The calculated settlement profile is shown in Fig. 12.

With only monument and fill present, the differential settlement between monument and fill would have been about 100 mm. With dewatering alone, there would have been no differential settlement. However, the reality is presence of monument and fill plus dewatering, and the observations are total and differential settlements of 2+ m and 328 mm, respectively. The key is that had the load been from only the monument and fill, all of the imposed stress would have acted in the re-loading stress range, but, because of the dewatering, the soil underneath the monument went into the virgin stress range, trebling the settlement for the monument as opposed to the settlement for the benchmark.

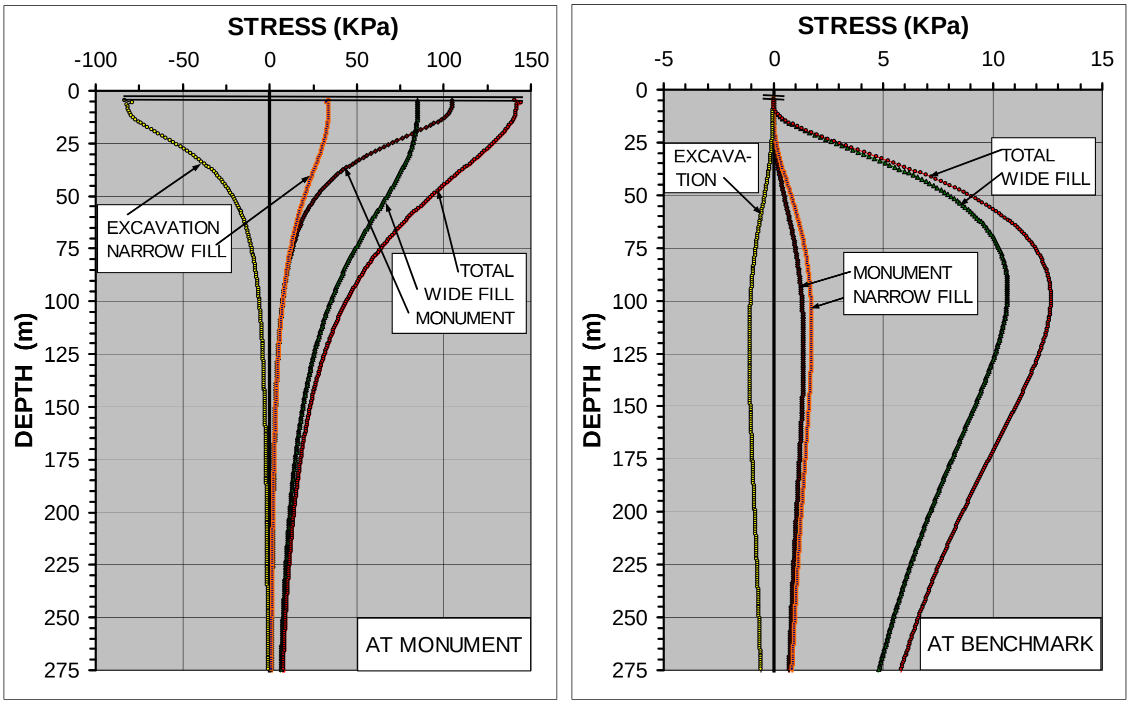

Fig. 11. Distribution of stresses from the monument, narrow fill, wide fill, and excavation at the monument and at the benchmark

The values of compressibility parameters and preconsolidation margins fit the observed settlements. But, so would several other combinations of input values. Therefore, we do not suggest that the values are representative for the Beaumont and Montgomery clays at the San Jacinto Monument site to the extent that they also would predict the settlement for other loading conditions and other sites. However, we do suggest that an analysis of the subsidence and the differential settlement at the San Jacinto Monument site as well as at other sites in the general area needs to include both the virgin and preconsolidation compressibility of the soil, and that the pore-pressure reduction due to the mining of the groundwater must be combined with the imposed loads.

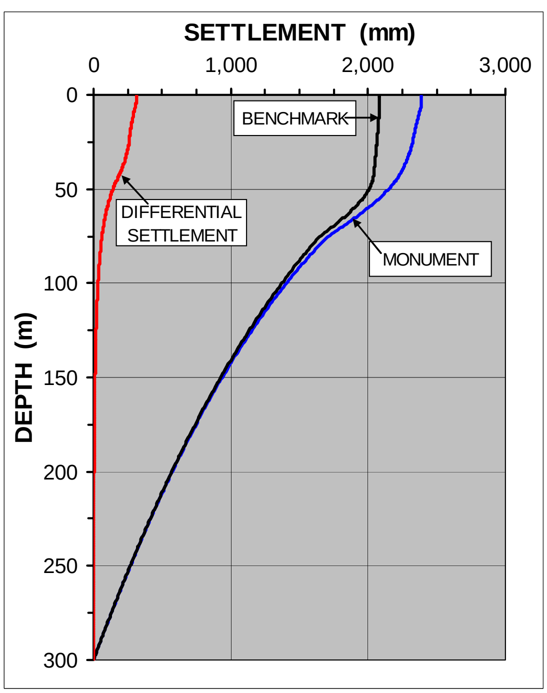

Fig. 12 Distribution of calculated settlement at the monument and benchmark, and measured differential settlement

On a final note, we trust that the monitoring of the monument settlement is continuing and will continue over the years to come. We believe it would be very enlightening and valuable if the site conditions could be more closely established. For example, by pushing a deep CPTU sounding, obtaining undisturbed soil samples for consolidation tests, and, most important, installing a set of piezometers to monitor the pore pressure distribution at the site.

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As an aside, the following figure presents the complete well records available from the Greater Houston, Texas, area for comparison to Figs. 7 and 8.