The Washington Monument Case History

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ABSTRACT: *The Washington Monument was built between 1848 and 1884 in honor of George Washington, first President of the United States of America. The Monument consists of a 169.16 m high column with a large pyramidal foundation. It was built in two periods straddling the Civil War. During the first period from 1848 to 1858, the foundation base was 24.38 m square, the column was built to a height of 55.5 m and the average pressure under the foundation reached 513 kPa. In 1879, the foundation was underpinned and the base of the foundation became a square ring with an outside dimension of 38.54 m and an inside dimension of 13.41 m. The column was completed in 1884 and created an average pressure under the foundation of 465 kPa.*

The original foundation rested on a medium compact sand - stiff clay mixture which was 3.76 m thick, underlain by a 8.30 m thick layer of very dense sand, gravel and clay, followed by a 11.68 m thick layer of stiff to very stiff high plasticity blue clay resting on decomposed Wissahickon schist. The underpinning brought the foundation level down to the very dense sand and gravel layer. Calculations indicate that the Monument settled about 1.3 m during the first construction period, 1848-1858. During underpinning in 1879, the measured settlement was 52 mm. During the completion of the column from 55.5 m to the full height of 169.16 m the measured settlement was an additional 63 mm. From 1884 to 1993, the Washington Monument settled an additional 55 mm in a very linear fashion with time. This remarkable case history is described in some detail including settlement and bearing capacity calculations which show that the underpinning process in 1879 likely saved the Monument from serious trouble.

KEYWORDS: Mat Foundation, Settlement, Consolidation, Case History

SITE LOCATION: IJGCH-database.kmz (requires Google Earth)

INTRODUCTION

The Washington Monument (Fig. 1) was built in honor of George Washington, first president of the United States of America. The construction took place over a long period of time (1848 to 1884) for various reasons including disagreement on what it should look like, construction problems, running out of money, and the Civil War. The settlement of the structure was recorded starting with the underpinning in 1879 until the last reading in 1993. This record represents very valuable information which can be used to estimate the settlement during the first phase of construction as well as evaluate the current calculation techniques. The data became available to the authors in 2006 courtesy of Mueser Rutledge Consulting Engineers (MRCE) and was supplemented by documents from various libraries and intense internet searches. This article is a presentation of the case history in some detail and of the results of a consolidation settlement analysis and a bearing capacity analysis.

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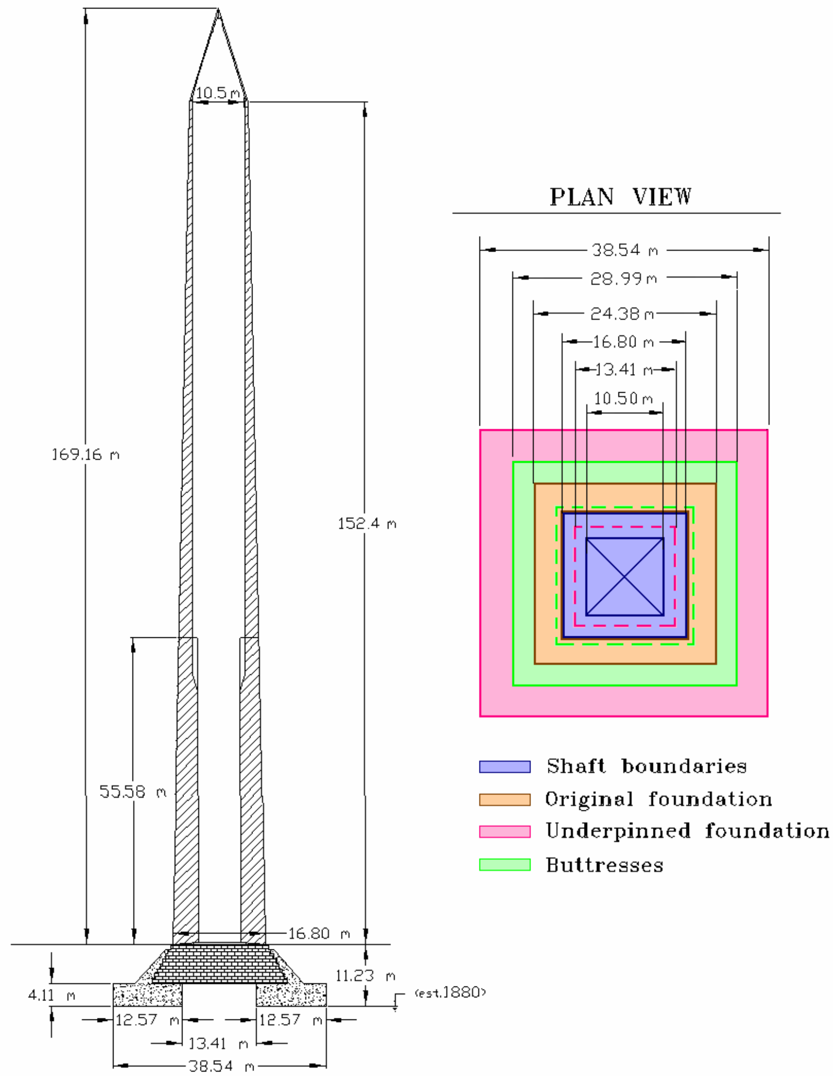

Figure 1. Dimensions of the Washington Monument.

HISTORY (NPS 2009)

The decision to honor the life of George Washington through a monument dates back to August 1783, when Congress unanimously authorized the erection of an equestrian statue of Washington. Its location was not decided until Pierre Charles L'Enfant, a French-born American architect and urban planner, was commissioned in 1790 to draw a plan for the new nation's capital: Federal City (modern day Washington D.C.). L'Enfant recommended placing the equestrian statue of Washington between the Capitol and the President's mansion. When George Washington died in 1799, the House of Representatives appointed a joint committee to decide on a more fitting memorial to honor Washington. This committee recommended that a marble monument be erected in the Capitol Building in Federal City, with Washington's body buried underneath it. The next thirty years saw disagreement and discussion as to the location and nature of the monument. In the early 1830s, citizens from the Federal City area formed the Washington National Monument Society with the goal of erecting a monument from the funds provided by the general public. In August of 1836, the Society solicited designs from American artists and architects, and selected the design of architect Robert Mills, who had previously designed a monument in Washington's honor in Baltimore. His recommended design featured a 76 m diameter colonnaded pantheon surrounding a 152 m high obelisk (Fig. 2a). Over the next 12 years the Society raised money, debated the merits of the proposed monument, and finally decided to construct only the obelisk due to limited funds. The location for the monument was based on the recommendations of L'Enfant's 1790 design, but swampy and unstable site conditions mandated that the new location be about 113 m East of the White House axis and 38 m South of the Capitol axis.

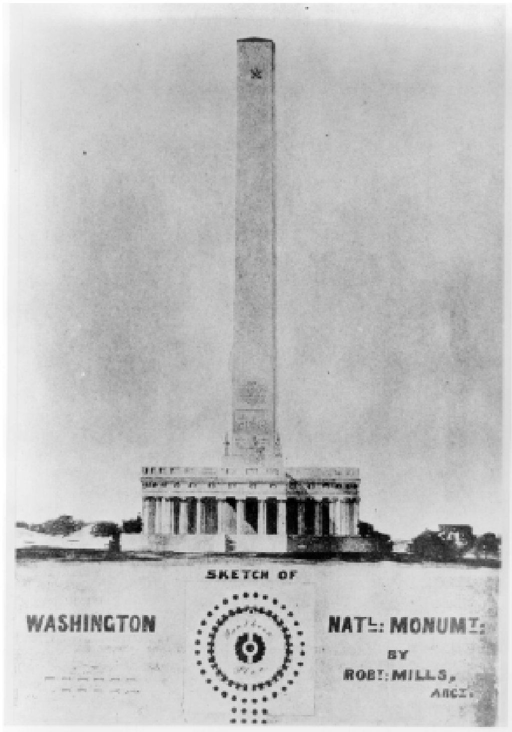

Figure 2a. Original design by Robert Mills. *Figure 2b. Final design – constructed monument.*
Figure 2. The Washington Monument (NPS, 2009).

GEOLOGY (U.S. Geologic Survey 1950)

The swampy soil conditions under the original location of the monument were largely caused by the nearby Potomac River and Goose (Tiber) Creek which subjected the area to tidal fluctuations and flooding. Fig. 3 is a redrafting of a 1792 land tract ownership map (U.S. Geologic Survey 1950), which shows the historic locations of the Potomac River and Goose Creek superimposed with a 2007 satellite image (Google Maps 2007).

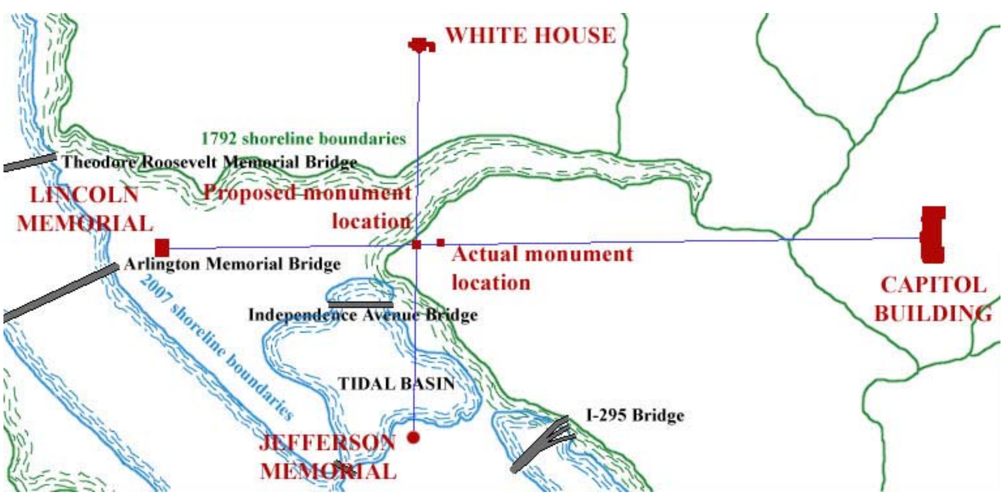

Figure 3. Overlay comparing shoreline boundaries from 1792 to 2007.

At the site, the Pleistocene terrace deposits lie immediately on top of the bedrock without the presence of the expected Cretaceous soils. This suggests that Pleistocene streams underlain by the shallow bedrock washed away the older Cretaceous soil. At the time of these stream deposits, the sea level varied greatly with abrupt climate changes, resulting in the presence of highly variable medium-plastic and slightly-to-moderately organic clay, sand, gravel, and sometimes boulders. Beneath these layers is the coastal plain formation of crystalline bedrock, which exists as decomposed schist, named Wissahickon Schist. This bedrock material is predominantly pelitic in nature with interlayers of quartzite. It is a metamorphic rock dating from the Ediacaran to the Cambrian age.

CONSTRUCTION (Torres, 1985)

Construction began under the supervision of architect Robert Mills in the spring of 1848 with a 2.34 m deep excavation to the foundation level. The foundation structure is a stair stepped pyramid made of blocks of blue gneiss, quarried from the Potomac Valley. The blocks weighed 60 to 80 kN and were mortared together with a mixture of hydraulic cement, lime, and angular sand. The shaft is made of blocks of marble from Baltimore, which were tested to ensure that the compressive strength was sufficient to sustain the weight of the structure. By the end of 1858, the shaft had risen to a height of 55.5 m when the work stopped because money had run out. For various reasons including the Civil War, construction only resumed in 1879 after a board of engineers was appointed by the US Army Corps of Engineers to evaluate the status of the Monument. This Board determined that in its present state the weaker clay and sand layer immediately under the foundation was almost at the point of bearing capacity failure. They reported as much as 0.23 m of settlement since construction but later stated that they had used the wrong reference point when surveying the site so the exact amount of settlement is unknown for the first part of construction.

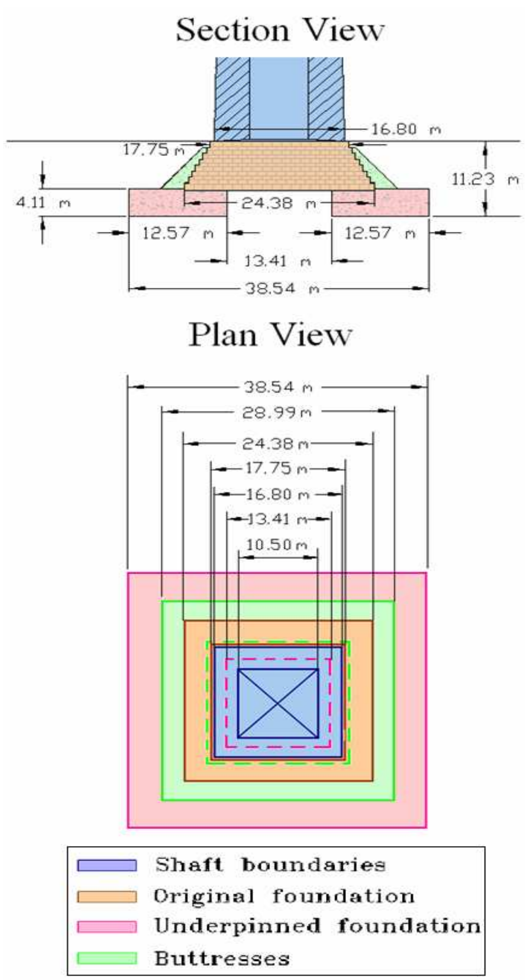

Figure 4. The two foundation systems (1848 and 1879).

The US Army Corps of Engineers put Lieutenant Colonel Casey in charge of completing the project. Casey decided that the foundation was inadequate and devised a method to underpin and widen it (Fig. 4). Underpinning consisted of digging 1.22 m wide trenches underneath the existing foundation to a depth of 3.76 m where lied the existing groundwater table. The intended bearing stratum for the underpinning was a very dense layer of sand and gravel. The earth removed from the trench was replaced with Portland cement concrete. The end result was a Portland cement concrete foundation shaped like a square ring underneath the existing gneiss foundation (Fig. 4). Left inside the new foundation was the intact natural soil which was not removed during underpinning. In 1880-1881, a terrace was constructed by moving earth around the foundation. This brought the ground level up to the top of the foundation. Then construction of the shaft resumed and was completed to a height of 169.16 m above the top of its pyramidal foundation. The Monument was completed in 1884.

GEOMETRY, WEIGHT, LOADING

The Washington Monument is a classical Egyptian-style four-sided obelisk topped by a pyramidion. The shaft rises 152.4 m above the present ground surface and is topped by the pyramidion for a total height of 169.16 m. At its base, the shaft is 16.8 m wide with a wall thickness of 4.58 m. At the top, the width is 10.5 m and the wall thickness is 0.46 m. The original pyramidal gneiss foundation is 17.83 m wide at the top and 24.38 m wide at the bottom. The total thickness of the original foundation is 7.16 m. The underpinned foundation has a square ringed shape with an outside width of 38.54 m and an inside width 13.41 m. The height of the underpinning is 4.11 m. The underpinning effectively increased the area of the foundation from 595 m^2 to 1486 m^2 when considering the outer square as the limit of the foundation. Fig. 4 shows a cross-section and plan view of the dimensions of the foundation.

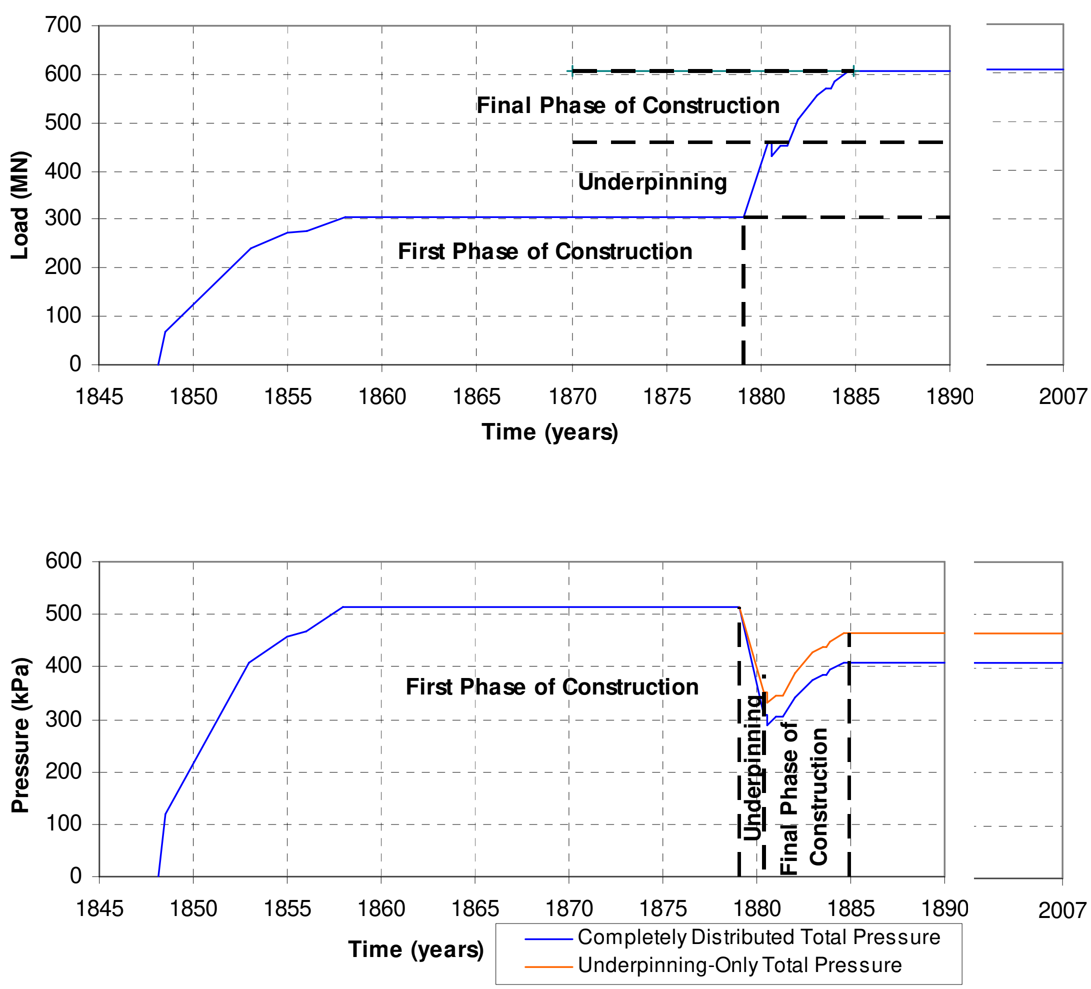

Figure 5. Evolution of load and pressure during construction.

The weight of the initial gneiss pyramidal foundation is 70 MN which created a pressure of 118 kPa (MRCE 1973). The weight of the Washington Monument at the end of the first construction period (1858) including the foundation was 305 MN which created a pressure of 513 kPa; at that time the Monument was 55.5 m high. The underpinning added 153.8 MN of foundation weight but reduced the pressure because of the increased foundation area. At the end of construction, the weight was 607.7 MN. By comparison, the San Jacinto Monument weighs 313 MN (Briaud et al. 2007), the Tower of Pisa 142 MN (Jamiolkowski 2006), and the Eiffel Tower 94 MN (Sigdel 2007). Note that the foundation of the Washington Monument represents 37% of its total weight. Fig. 5 shows the evolution of the total load as a function of time during the construction period. When calculating the average pressure under the foundation, two assumptions can be made. One is that the total load is distributed equally over the entire area of 38.54 x 38.54 m = 1486 m^2 including the soil plug under the center of the underpinned foundation. This leads to a pressure of 409 kPa. Another assumption is that the total load is distributed only over the area associated with the foundation's underpinning, a square ring with an area of 1306 m^2. This leads to a pressure of 465 kPa. Fig. 5 shows the evolution of the total average pressure immediately under the monument foundation during the period of construction. For this graph, after underpinning was completed (1880), two total pressures are presented from the assumption defined above.

In July of 1880, an earth terrace was placed around the foundation of the monument. This terrace was 5.2 m (17 ft) high, raising the ground surface to the top of the original foundation and buttresses, and 9.1 m (30 ft) wide. This terrace was expanded laterally in 1881, maintaining the same height but adding fill around the monument in the shape of an ellipse, measuring a minor diameter of 47.2 m (155 ft) and a major diameter of 67 m (220 ft). The weight of the terrace is estimated to add a pressure of 86.4 kN/m^2 around the monument, assuming a unit weight of the soil of 16.6 kN/m^3, which comes from an average of unit weights at these depths that were found during prior studies (MRCE 2002). These terraces exist today in the same dimensions, with the exception of minor amounts of re-grading in 1977 and 2003.

SETTLEMENT MONITORING

The settlement of the Washington Monument has been monitored since February 1879, coinciding with the beginning of the foundation's underpinning under the leadership of Lt. Col. Thomas Casey (Olszewski 1971). The settlement was monitored to ensure that the invasive procedure of channelized underpinning did not tilt the shaft. Casey placed reference points at each corner of the top of the original foundation (MRCE, 2002). As a second indicator of the monument's tilt, a twenty pound plumb bob was suspended from the southwest corner of the shaft. During underpinning, settlement readings for each corner were taken and recorded once daily, and the relative position of the plumb bob was recorded twice daily. For the time period associated with the underpinning of the monument, the average settlement of the four corners was 52 mm (MRCE 2002).

Shortly after the foundation's underpinning was complete, concrete buttresses were placed to provide lateral support to the original foundation. These buttresses covered all four reference points originally placed on the foundation, but Lt. Col. Casey continued to document the settlement of the monument in his monthly reports without a description of the reference points he was using. A record of all settlement readings since underpinning begun is presented in Fig. 6. Note that these settlement readings are all relative to a value of zero settlement corresponding to the beginning of underpinning.

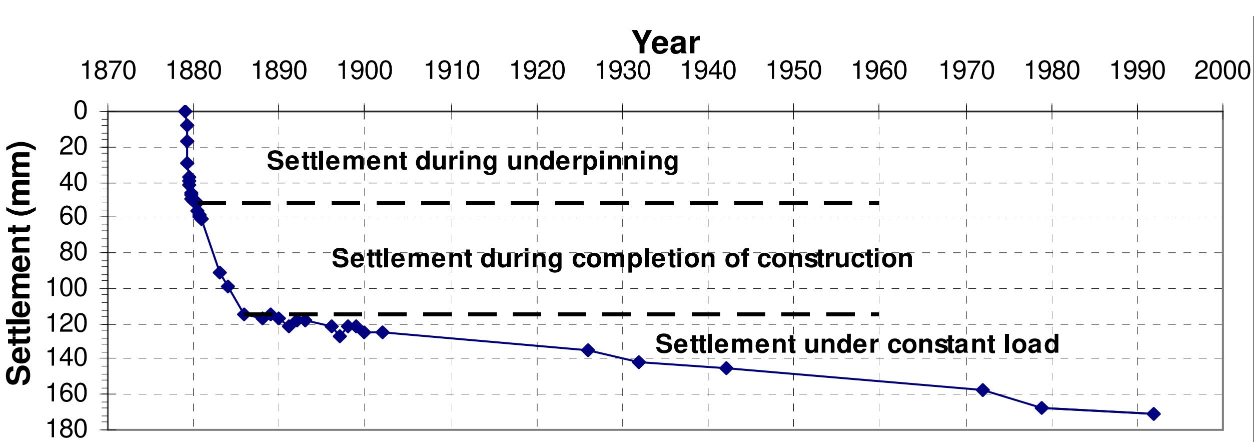

Figure 6. Settlement of the Washington Monument since underpinning (MRCE, 2002).

The Mueser, Rutledge, Johnson, & Desimone report from 1984 describes the source of the survey points from the completion of the monument until 1979. The settlement readings in 1886, 1902, 1926, and 1932 were published by the Special Committee on Earth and Foundations in the ASCE Proceedings of May 1933. The settlement readings in 1898, 1900, and 1930 were documented by Darton in the Geological Survey Professional Paper #217, where he stated that the soil had permitted 140 mm of sinking from 1879 to 1898, but with the widened base, the amount of settlement from 1900 to 1930 was only about 15 mm. Darton (1950) makes no mention of the benchmark, the settlement reference points, or the organization that performed the measurements. The readings in 1923, 1930, 1942, 1972, and 1979 were taken by the U.S. Coast and Geodetic Survey. These readings refer back to the Meridian Stone, a vertical control benchmark monumented and standardized in 1890. The Meridian Stone is located at the center of the ellipsoid in front of the property line of the White House, 90 m west and 459 m north of the center of the Washington monument. It is marked by a bolt in the center of a square granite stone that is set flush with the ground (NGS 2009). The exact embedment of this bench mark is unknown but it is likely that the embedment is very shallow.

SOIL BORINGS, SOIL STRATIGRAPHY, GROUND WATER LEVEL

Studies near the Monument in 1930, 1962, 1973, and 2002 have yielded a total of 51 borings (Fig.7). Table 1 summarizes the borings performed, the depths to which the borings were completed, the observed depth to groundwater, and the intended use of this information. A generalized stratigraphy for the soil underneath the Washington Monument is presented in Fig. 8. The fill material from the terraces, constructed in 1880 and 1881, was placed around the Monument's foundation. Below the fill is a layer of medium compact sand and stiff clay within which the original foundation rested. Underneath this layer lays a south-southeasterly sloping deposit of interbedded lenses of very dense sand, gravel and clay on which the underpinned foundation rests. Beneath these interbedded layers lies a layer of varying thickness of stiff to very stiff plastic moderately organic blue clay with occasional organic lenses, which also slopes to the south-southeast. Below the stiff to very stiff clay layer is the Wissahickon Schist bedrock which slopes gently to the south- southwest.

Table 1. Borings performed near the Washington Monument.

Date	Number of Borings	Depth of Borings	Groundwater Elev. (from MSL)	Reason for borings
	20	3.65 m	N/A	Vehicle barrier system retaining walls
2002	5	15.25 m	+3 m to -3 m	For proposed visitor's facility
	2	27.24 m	+0 m to -3 m	For proposed visitor's facility
1973	2	29.5 - 34.2 m	- 3.35 m	For proposed interprative facility
1962	7	27.4 - 30.4 m	-2.75 to -3.65 m	Loading limitations around the site
1930	7	36 -38.5 m	- 3 m	Inner ring borings - regrading limitations
1930	8	36 - 38 m	- 3.1 m	Outer ring borings - regrading limitations

Analysis of the boring data indicates that the water table slopes gently from the west to the east with an average depth of -3.05 m below mean sea level or 3.65 m below the bottom of the underpinned foundation, or 13 m below the ground surface. Historical documents suggest that the groundwater level was at a level 152.4 mm above the bottom of the underpinned foundation (Parkhill 1957). Therefore since 1880, a minimum of 3.65 m groundwater drawdown seems to have taken place as a result of the development of the Mall area, the depressed roadways, and the subway construction. USGS data from a well approximately half a mile from the Monument shows drawdown has occurred and is continuing to occur (Fig. 9). This change in water level may have an effect on the relative settlement of the Monument (Fellenius and Ochoa 2009). For example, a decrease in water level of 3.65 m over that period of time will create a gradual increase in effective stress in the soil below equal to about 36 kPa. This could equate to approximately 75 mm of additional settlement. On the other hand, one would think that, since the draw down is on a much larger scale than the Washington Monument, and since the nearby benchmark (Meridian Stone) is likely not deeply embedded, the Monument is subjected to the same settlement due to drawdown. As a result, it is thought that the settlement of the Monument due to the drawdown of the water table is not included in the settlement of the Monument recorded with respect to the benchmark.

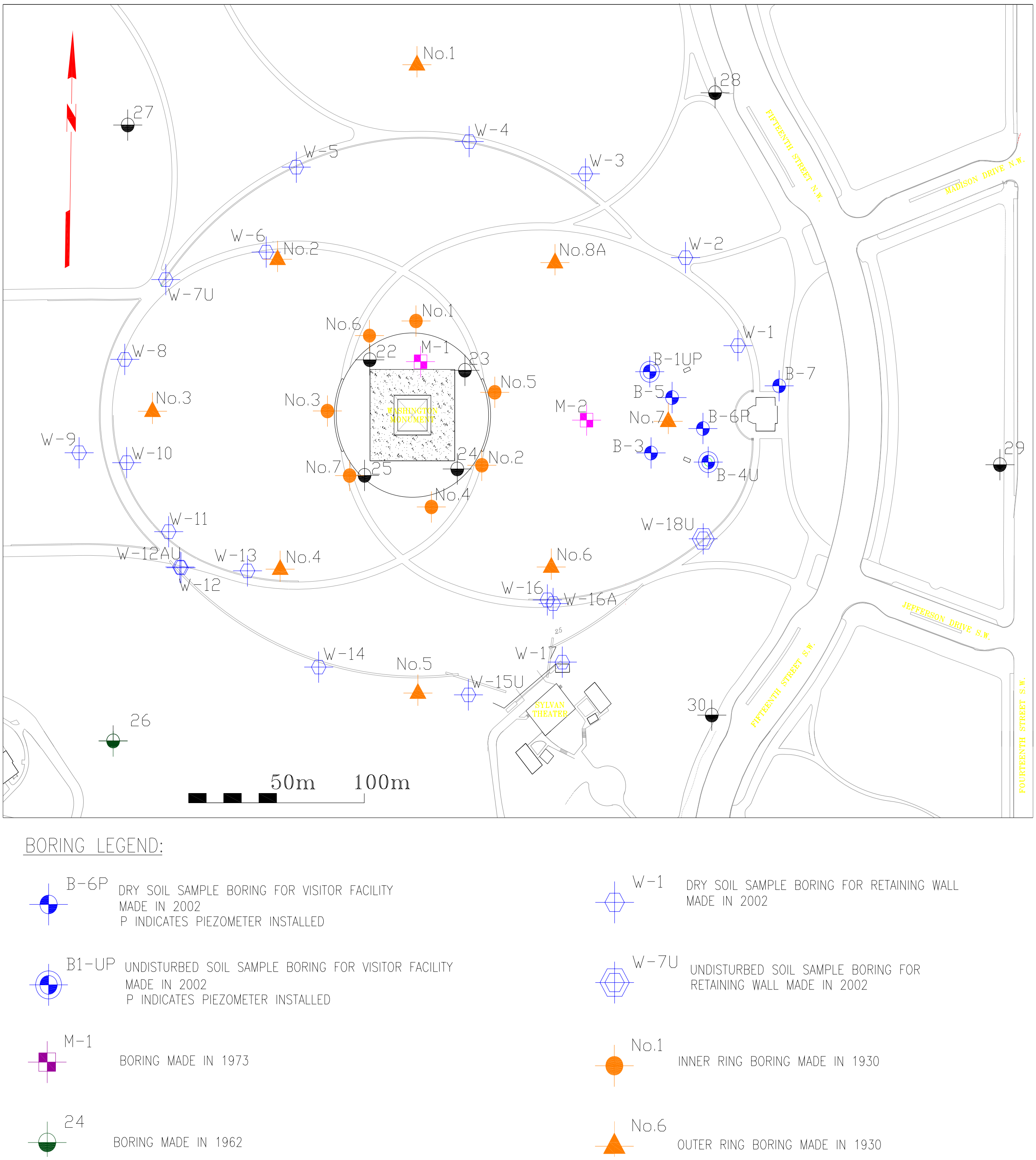

Figure 7. Boring locations around the Washington Monument (after MRCE, 2002).

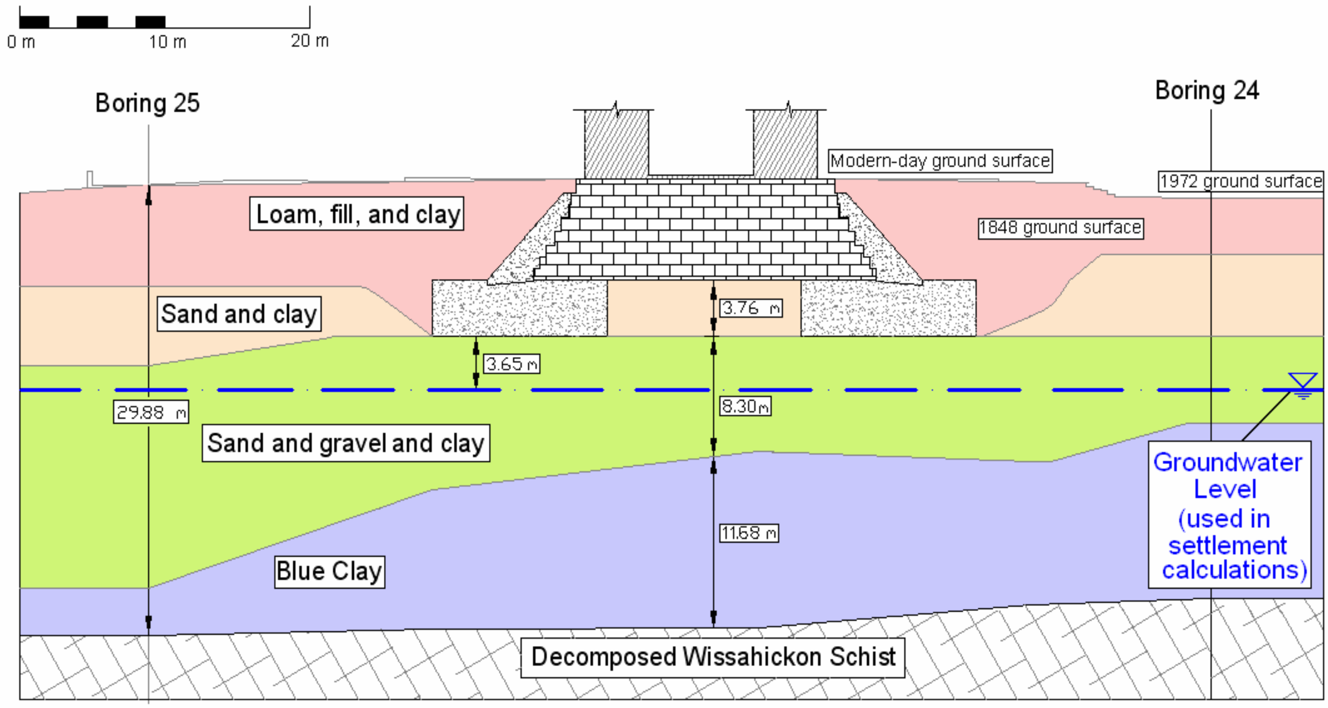

Figure 8. Generalized stratigraphy under the Washington Monument.

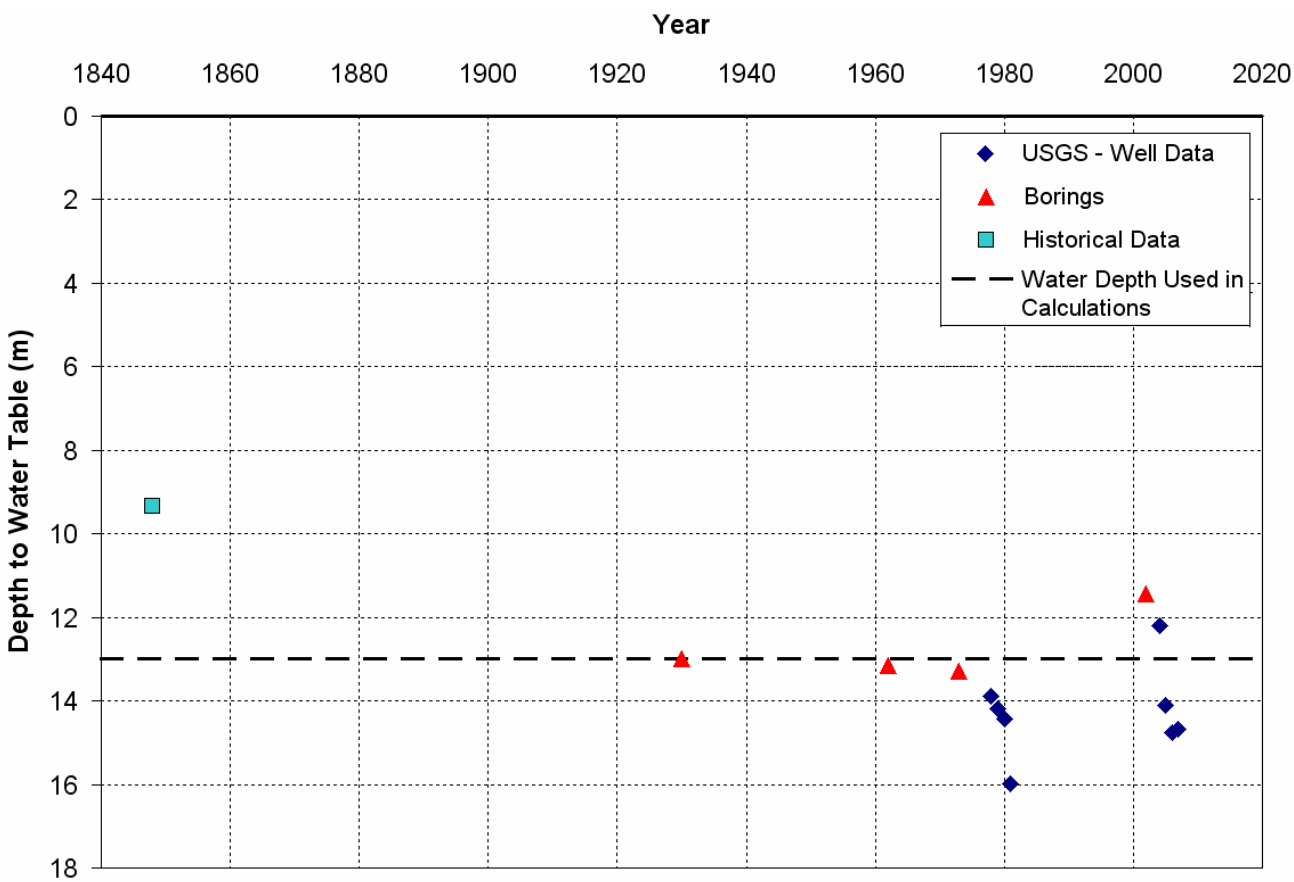

Figure 9. Water Table Data around the Washington Monument.

SOIL PROPERTIES

The index properties available from the data obtained since construction are: the total unit weight (γ_t) averaging 17 kN/m^3, the natural water content (w) (Fig. 10A), and the plasticity index (PI) (Fig. 10B). The total unit weight was found in the 1962 borings. The natural water content profile comes from a combination of the seven 1962 borings, the twenty-seven 2002 borings, and the 1973 borings (within the clay layer only). The plasticity index profile comes from the 1973 borings.

Standard penetration (SPT) tests were also conducted in borings in 1962, 1973, and 2002. Fig. 10C shows the blow count profile for the borings located close to the foundation of the monument. Note the low blow counts under the old foundation and the very high blow counts under the underpinned foundation. The undrained shear strength of the stiff to very stiff plastic blue clay layer was measured in 1962 and 1973 by unconfined compression tests, and in 2002 by pocket penetrometer tests and torvane tests. Fig. 10D shows a profile of the undrained shear strength of the soil obtained from those tests around the monument with respect to depth.

The consolidation properties were obtained from Barber's 1962 report. Barber reports the results of 36 consolidation tests at varying depths from samples collected close to the foundation of the monument. Barber gives preconsolidation pressures in the range of 380 to 475 kPa. Recalculating these preconsolidation pressures using Casagrande reconstructions for each of the consolidation tests gives a range of 50 to 150 kPa in the shallow clay directly below the original foundation and 150 to 800 kPa in the deeper clay, with an overall average of 365 kPa (Fig. 10G). These consolidation curves were used to obtain the compression index Cc (Fig. 10E) and the recompression index Cr (Fig.10F). For each test, two values for Cr were obtained: one from the slope at the beginning of the consolidation curve between the first stress point and the second stress point labeled Cr and the second from the slope of the unload-reload cycle performed in the consolidation test (Fig. 10F). The coefficient of consolidation c_v is presented in Fig. 10H.

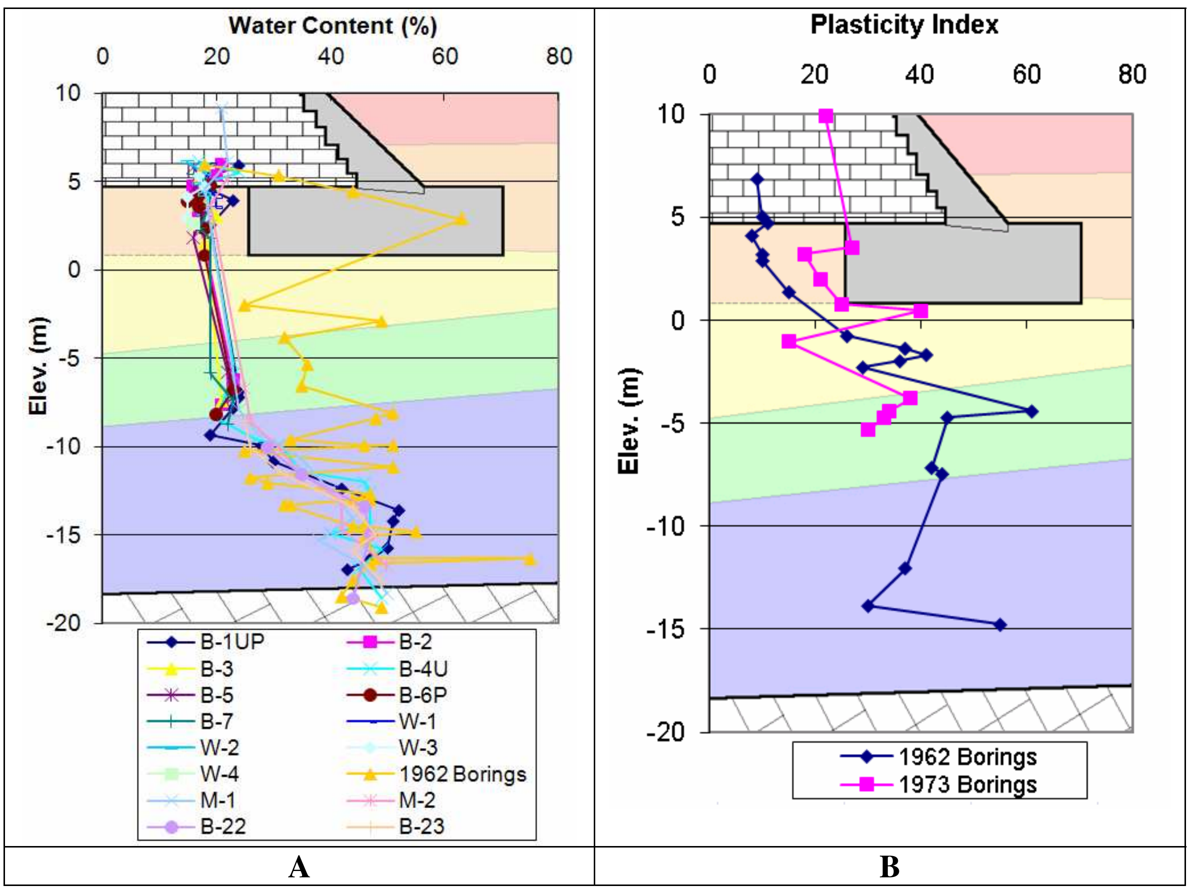

Figure 10. Properties of the soil under the Washington Monument. A) Water content, B) Plasticity index.

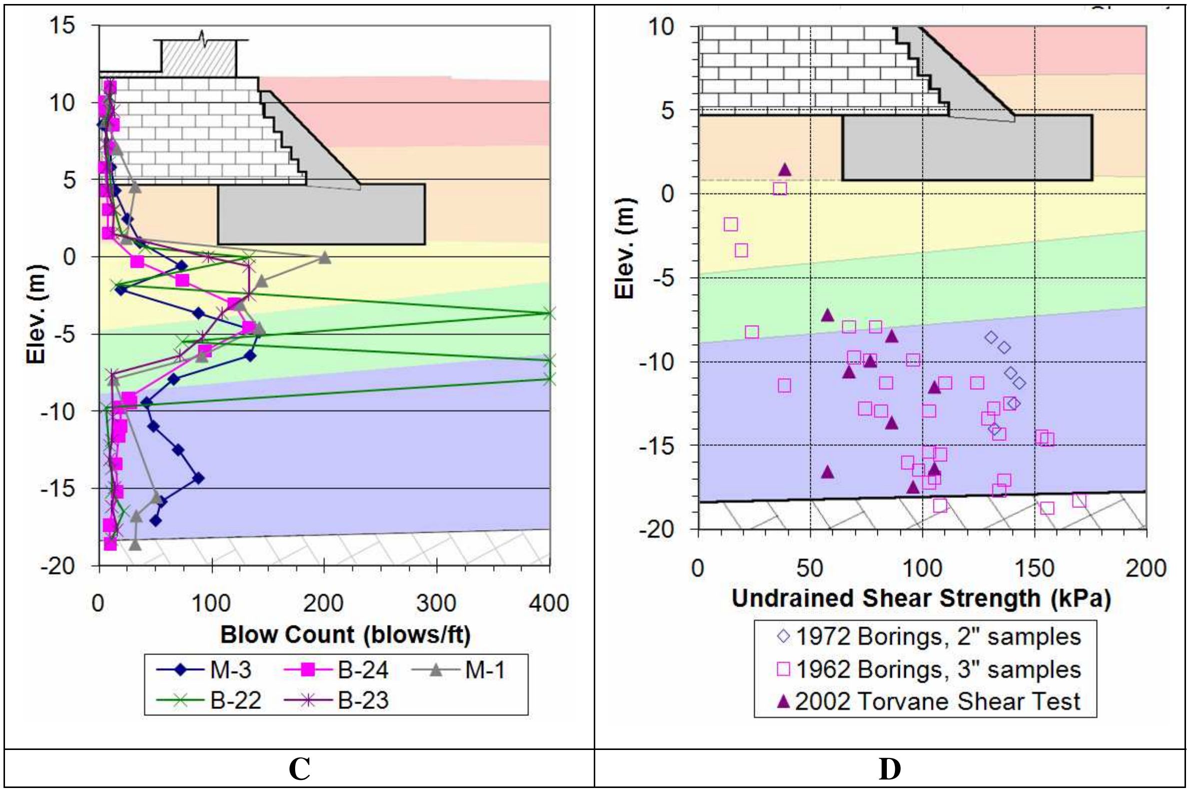

Figure 10. Properties of the soil under the Washington Monument. C) SPT blow count, D) Undrained shear strength.

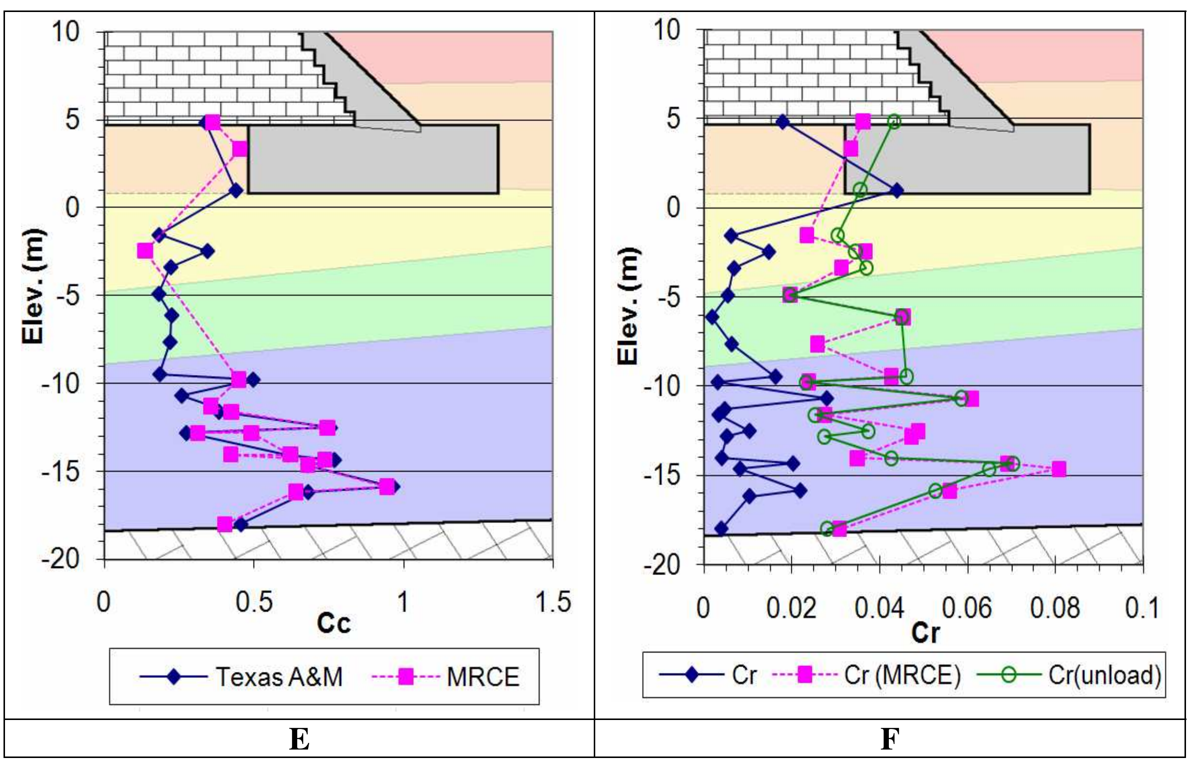

Figure 10. (ct'd) – Properties of the soil under the Washington Monument. E) Compression index, F) Recompression index.

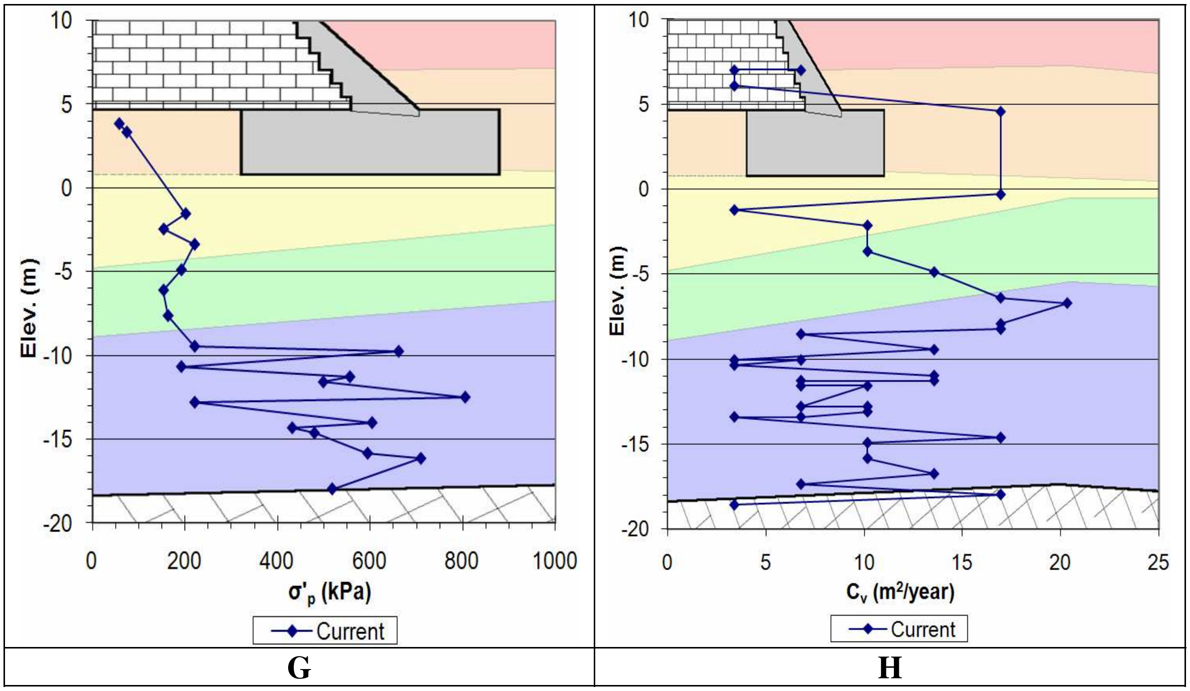

Figure 10. (ct'd) – Properties of the soil under the Washington Monument. G) Preconsolidation pressure, H) Coefficient of consolidation.

ULTIMATE BEARING CAPACITY

Before underpinning, the foundation was resting on a gently sloping layer of medium compact sand and stiff clay averaging 3.78 m of thickness above much stronger layers of sand and gravel and then clay. The most likely bearing capacity failure mechanism is a local bearing capacity failure at the edges of the gneiss foundation by squeezing of the weak sand-clay layer outwards away from the center. At the time of maximum loading with this foundation, the embedment depth was 2.34 m. If the sand and clay layer is considered to be primarily clay, then the ultimate bearing capacity is:

$$P_u = N_c S_u + \gamma D \tag{1}$$

Where P_u is the ultimate bearing pressure, N_c the bearing capacity factor, S_u the undrained shear strength, γ the total unit weight, and D the embedment depth. An undrained shear strength (S_u) of 72 kPa for that weak layer was obtained by correlation with the average SPT blow count of 12 blows/ft (Kulhawy and Mayne 1990). The value for N_c was taken as 6.2 for a square foundation (Skempton 1951). This gives a value for the ultimate bearing capacity of 491 kPa. If the sand and clay layer is considered to be primarily sand, then the ultimate bearing capacity is given by (Briaud and Gibbens 1999):

$$P_u\left[kPa\right] = 75 \times N\left[\frac{blows}{ft}\right] \tag{2}$$

Where N is the average SPT blow count below the foundation level. Using this correlation and the average blow count of 12 bpf, the ultimate bearing capacity of the soil underneath the foundation is 900 kPa. Therefore the ultimate bearing capacity of the foundation is in the range of 491 kPa to 900 kPa. Since the average total pressure due to the shaft and foundation during this time period was 513 kPa, it is very likely that the significant settlement calculated after the first phase of construction, and noted by the U.S. Corps of Engineers in 1878, was due to the foundation pressure approaching the ultimate bearing capacity of the underlying layer.

The monument's underpinning lowered the foundation level to the dense sand and gravel layer located underneath the stiff clay and medium compact sand layer. This new bearing stratum has an average SPT blow count of about 125 bpf, in high contrast to the material above (MRCE 1973). For the underpinned foundation, there are two possible bearing capacity failure mechanisms. The first failure mechanism is a bearing failure in sand layer upon which the underpinning is placed. For this soil layer, the average blow count of 125 bpf gives an ultimate bearing capacity of 9375 kPa according to Eq. 2. The second failure mode is a punching failure through the 9.3 m thick sand layer into the stiff blue clay layer below. The ultimate load to fail the foundation vertically in this case is the addition of the ultimate load necessary to fail the clay layer in bearing plus the ultimate load necessary for the foundation to punch through the sand layer:

$$P_u A_f = P_u\left(clay\right) A_f + \left(p_{inside} + p_{outside}\right) H \times k_o \sigma_{ov}^{'} \tan\phi \tag{3}$$

Where A_f is the area of the foundation in contact with the soil, P_u (clay) is the ultimate bearing pressure of the clay below, p_{inside} and $p_{outside}$ are the inside and outside perimeters of the foundation respectively, H is the thickness of the sand layer through which punching occurs, k_o is the coefficient of earth pressure at rest in the sand layer, σ'_{ov} is the vertical effective stress at the middle of the sand layer, and φ is the effective stress friction angle of the sand layer. The average undrained shear strength of the lower clay layer is taken from the profile as 100 kPa (Fig. 10D). The depth of embedment D was 15.5 m from the ground surface elevation before the fill was placed to the top of the lower clay layer. The width of the footing B was 12.57 m taken as the outside width minus the inside width of the underpinning divided by 2. With these parameters, the D/B ratio was 1.23, the corresponding N_c value, assuming a strip footing, was 6.65, and the ultimate bearing pressure for the lower clay layer was 959 kPa. If the k_0 value for the sand layer above is taken conservatively as 0.50, and the friction angle as 41 degree (from N = 125 bpf), then the contribution from the foundation punching through the sand layer gives an equivalent pressure of 28 kPa. Therefore the ultimate bearing capacity for this failure mechanism was 987 kPa which leads to a satisfactory factor of safety of about 2.4 against bearing capacity failure since the pressure at the end of construction was 465 kPa. Note that the bearing capacity equations were not available to Lieutenant Colonel Casey in 1878 and it is much to his credit to have reached such a satisfactory factor of safety without that knowledge.

DEPTH OF INFLUENCE AND INCREASE IN STRESS VERSUS DEPTH

The depth of influence is commonly defined as the distance from the foundation to the point where the pressure has decreased to 10% of the applied mean foundation pressure, or the depth at which the settlement is 10% of the foundation settlement. These two definitions are quite different (Briaud et al. 2007). In the case of the Washington Monument, the depth of influence is set by the presence of the shallow bedrock and the entire layer of soil between the monument's foundation and the bedrock is considered in the settlement analysis.

The distribution of stresses underneath the monument was evaluated using 3D elastic finite element modeling with ABAQUS. A uniform soil was assumed and a soil modulus of 10.6 MPa was selected. The modulus value for the soil was backcalculated using the theory of elasticity (Eq. 4) applied for the last phase of construction (August of 1880 – present day).

$$E = I \times \frac{\Delta p \times \left(B_1 - B_2\right)}{\Delta s} \tag{4}$$

The increase in pressure (Δp) on the soil, the settlement directly associated with this increase in pressure (Δs), Poisson's ratio (υ), and the average thickness of the compressible layer (h) are all known. Equation 3 applies to a flexible footing on a compressible layer of thickness underlain by a rigid base (Poulos and Davis 1974). It has been found that the distribution of stresses which gives the best predictions for a stiff foundation is the one obtained when assuming that the foundation is flexible (Briaud et al. 2007). It is argued that the distribution under the rigid foundation while theoretically correct for an elastic material does not correspond to the long term distribution of stresses under the foundation. The reason is that in the long term the soil redistributes the stresses to the point where the pressure distribution is uniform (Focht et al. 1978), a case consistent with a flexible foundation. The stress increase under the foundation of the Washington Monument was simulated for many different cases (Smith 2007). Fig. 11A shows the stress increase under the center A of the first foundation with the column one third of the way up (1858). Fig. 11B shows the stress increase under the center B of the periphery of the underpinned foundation with no additional load (1880) and then with the completed column load (1884). As can be seen the increase in vertical stress at completion (1884) was less than the one at the end of the first phase (1858); therefore the underpinning achieved the purpose of lowering the stress in the soil.

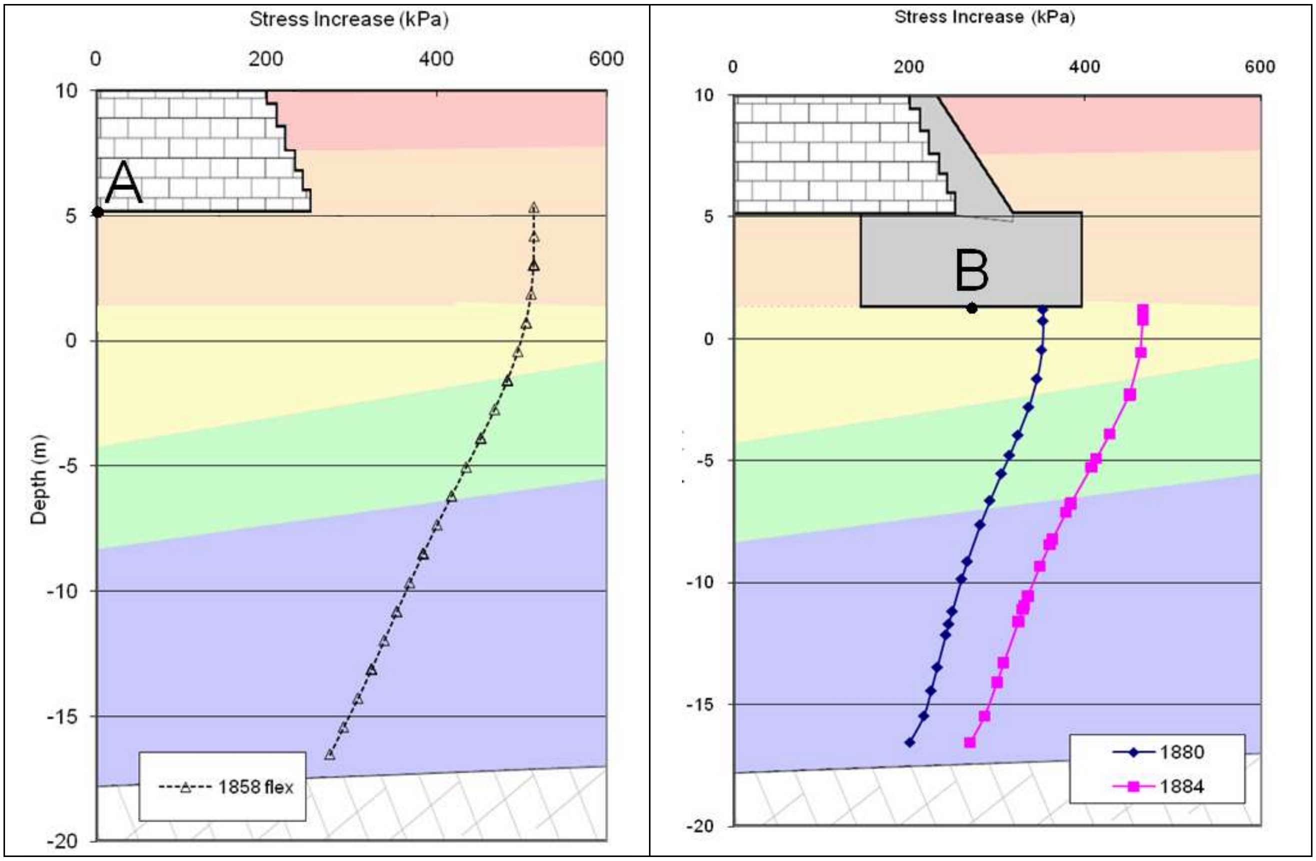

Figure 11. Increase in stress: A) below the center A of the 1858 foundation (flexible case), B) below the center B of the underpinning (flexible case).

CONSOLIDATION SETTLEMENT CALCULATIONS

Terzaghi's classical one-dimensional consolidation theory is used to calculate the settlement of the Monument. For these calculations, the zone of influence is defined as the depth to bedrock. The zone of influence is then broken into smaller layers. The depths and thicknesses of each of these layers correspond with the depths of the consolidation tests that Barber performed in 1962. Three historical phases are considered for settlement calculations: Phase 1 is from the beginning of construction through the pause in construction during the Civil War (1848 – 1878), Phase 2 is the underpinning of the monument, with no progress made on the shaft. (1879 – 1880), and Phase 3 is the completion of the monument to its current state (1880-present). A settlement calculation is performed for Phase 1 (Point A on Fig.11A under the center of the original foundation) and for Phase 3 (Point B on Fig. 11B under the center of the underpinning foundation). During Phase 1, the vertical effective stress within the depth of influence increases from the initial vertical effective stress, σ'_{ov}, to $\sigma'_{ov} + \Delta\sigma_{v1}$ where $\Delta\sigma_{v1}$ is the increase in vertical stress due to load given by Fig. 11A. During Phase 2 a theoretical rebound takes place when the effective vertical stress decreases from $\sigma'_{ov} + \Delta\sigma_{v1}$ to $\sigma'_{ov} + \Delta\sigma_{v2}$ where $\Delta\sigma_{v2}$ is given by Fig 11B for the 1880 profile. During Phase 3 further settlement takes place as the vertical effective stress increases from $\sigma'_{ov} + \Delta\sigma_{v2}$ to $\sigma'_{ov} + \Delta\sigma_{v3}$ where $\Delta\sigma_{v3}$ is given by Fig. 11B for the 1884 profile corresponding to the end of construction.

Two methods are used to calculate the settlement. The first method, called the equation method, involves the following equations:

$$\Delta H_i = \frac{H_o}{1+e_o}\left[C_c \log\left(\frac{\sigma'_{ov} + \Delta\sigma_v}{\sigma'_{ov}}\right)\right] \text{ when } \sigma'_p \le \sigma'_{ov} \tag{5}$$

$$\Delta H_i = \frac{H_o}{1+e_o}\left[C_r \log\left(\frac{\sigma_p^{'}}{\sigma_{ov}^{'}}\right) + C_c \log\left(\frac{\sigma_{ov}^{'} + \Delta\sigma_v}{\sigma_p^{'}}\right)\right] \text{ when } \sigma_{ov}^{'} < \sigma_p^{'} < \sigma_{ov}^{'} + \Delta\sigma_v \tag{6}$$

$$\Delta H_i = \frac{H_o}{1+e_o}\left[C_r \log\left(\frac{\sigma_{ov}^{'} + \Delta\sigma_v}{\sigma_{ov}^{'}}\right)\right] \text{ when } \sigma_{ov}^{'} \le \sigma_{ov}^{'} + \Delta\sigma_v \le \sigma_p^{'} \tag{7}$$

Where ΔH_i is the compression of one layer, H_o the thickness of that layer, e_o is the initial void ratio of that layer, C_c is the compression index, C_r is the recompression index, σ'_{ov} the initial vertical effective stress at the center of the layer, $\Delta\sigma_v$ the increase in stress at the center of the layer, and σ'_p the preconsolidation pressure. The consolidation indices C_c and C_r were obtained from Barber's 1962 consolidation curves. As described before two C_r values were determined, one from the slope at the beginning of the consolidation curve between the first stress point and the second stress point labeled C_r and the second from the slope of the unload-reload cycle performed in the consolidation test labeled C_r(unload) (Fig. 10F). Note that for the settlement calculations of Phase 1 the preconsolidation pressures are those of Fig. 10G but for the settlement calculations of Phase 3, the preconsolidation pressure was considered to be the effective vertical stress at the end of Phase 1 ($\sigma'_p = \sigma'_{ov} + \Delta\sigma_{v2}$). Therefore for Phase 3 the settlement was considered to be due mostly to reloading of the soil.

The second method, called the curve method, makes use of the strain values found on the consolidation curves for each corresponding stress value. For a given layer, the initial vertical effective stress, σ'_{ov}, is calculated and the corresponding strain is read on the consolidation curve for that depth. Then, the process is repeated for the final stress, $\sigma'_{ov} + \Delta\sigma_v$, and the corresponding strain value is obtained from the same curve. The compression of the layer is calculated as the difference between the initial and final strain times the layer thickness. For the initial loading (Phase 1) the virgin curve is used. For Phase 3, the following approach was taken. For a given layer, the vertical effective stress immediately before underpinning, $\sigma'_{ov} + \Delta\sigma_{v1}$, was determined, and an unloading line was drawn through the point corresponding to that stress on the consolidation curve. The selected slope for that unloading line was the one corresponding to Cr or Cr(unload) which ever was closest. The initial strain for the Phase 3 settlement calculations was the strain on that unload reload line at the stress $\sigma'_{ov} + \Delta\sigma_{v2}$. The final strain was the one corresponding to the final stress after completion of the Monument, $\sigma'_{ov} + \Delta\sigma_{v3}$. At some depths, this point was on the drawn unload line, but at larger depths, this point was on the virgin compression curve. Table 2 shows the labeling system for these cases and Table 3 shows the calculated settlements by the various methods.

Table 2. Definition of sub-cases.

Calculation Method	Sub-Case
Strain values read from consolidation curves	a
Closed-form settlement equations using Cc and Cr	b
Closed-form settlement equations using Cc and Cr(unload)	c

Table 3. Calculated settlement value.

Assumption Case	Settlement (m)		
	Sub-case		
	a	b	c
Phase 1 (calculated)	1.328	1.398	1.465
Phase 3 (calculated)	0.116	0.102	0.130
Phase 3 (measured)	0.119	0.119	0.119

Both methods rely on 1-D consolidation theory where no lateral strain occurs. The problem is more complicated, however, and includes factors such as lateral deformation, creep, 3-D effects and possible rebound during underpinning. While these factors are real (Skempton and Bjerrum 1957, Holtz and Kovaks 1981, Biot 1941), they were not included in this simple set of calculations as they are not met with complete agreement amongst practitioners. Instead, 1-D calculations were performed and compared to the measurements.

SETTLEMENT MEASUREMENTS

There is no accurate settlement record for the time period between the beginning of construction and the beginning of the underpinning of the monument (1848-1879). It is known that the US Army Corps of Engineers was sufficiently worried about the settlement and stability of the structure to warrant the underpinning effort (Abbot 1987; Olszewksi 1971; Torres, 1985). The board of engineers appointed by the US Army Corps of Engineers to evaluate the status of the Monument in 1876 reported as much as 0.23 m of settlement since construction but later stated that they had used the wrong reference point when surveying the site so the exact amount of settlement is unknown for the first part of construction (Torres, 1985). What is known is that it must have been significant enough to warrant a new foundation system. The calculated settlement presented here is over 1.3 m.

The settlement was observed and recorded starting at the beginning of the underpinning process. The underpinning of the Monument led to an average additional settlement of 52 mm. This settlement may be due to the continuation of the settlement under the Phase 1 load or to the disturbance created by the underpinning process (trenching and excavation) or both. The settlement which occurred between the end of the underpinning (1880) and the end of construction in 1885 was 63 mm. The settlement from 1885 and 1992 has been 55 mm. The 119 mm of settlement between the end of underpinning and 1992 agrees well with the calculated values in Table 3. The linear shape of the settlement curve from 1885 to 1992 would indicate that creep is contributing to the settlement.

CALCULATED VERSUS MEASURED SETTLEMENT RATE

The rate of settlement is predicted using Terzaghi's theory of one-dimensional consolidation (Holtz and Kovacs 1981).

$$T = \frac{t \cdot c_v}{H_{dr}^2} \tag{8}$$

Where T is the time factor, t is the time since consolidation began, c_v is the coefficient of consolidation, and H_{dr} is the minimum drainage length. The minimum drainage length was taken as 9.2 m, assuming one-way drainage in the blue clay layer. The average coefficient of consolidation, c_v, in that layer was determined from the consolidation tests by Barber in 1962 to be 10.2 m^2/year. The minimum c_v value is 3.39 m^2/year (Fig. 10H). Fig. 12 shows two predicted settlement curves along with the observed settlement curve from 1879 to 1993. A rate of consolidation, represented by the coefficient of consolidation (c_v), and a maximum settlement value were used to fit the two predicted settlement curves to the measured settlement curve. It is clear that there is a discrepancy between the theory and the observation. Fig. 12 shows that the measured settlement curve deviates from the consolidation theory starting around 1900. It is likely that creep settlement and general lowering of the ground water level are contributing to the difference between the consolidation theory and the measurements.

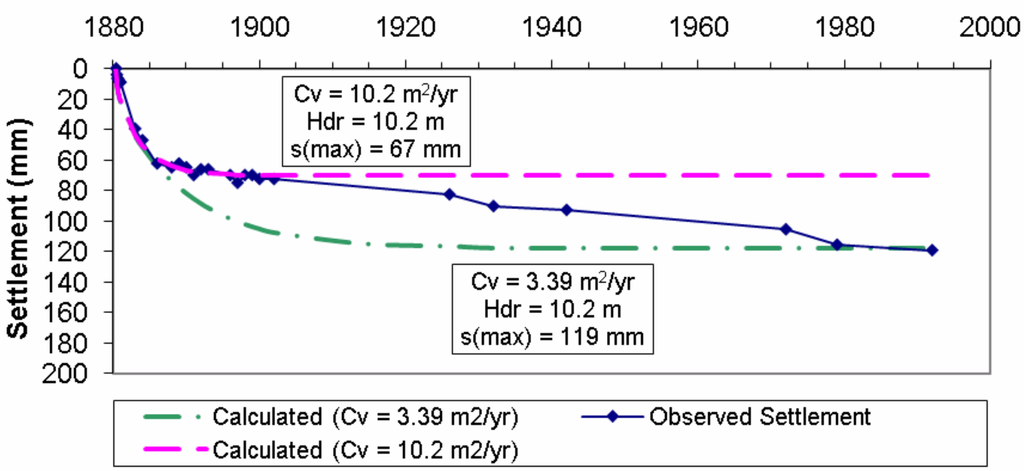

Figure 12. Calculated vs. predicted settlement.

The predicted settlement vs. time curve for the first phase of construction, using the same parameters as the ones used for the phase since underpinning, is shown in Fig. 13. Fig. 13 covers the entire life of the Monument. The settlement vs. time curve for the San Jacinto Monument in Houston, Texas is shown in Fig. 14 for comparison purposes (Nicks, 2005). Both the Washington Monument and the San Jacinto Monument are in locations that suffer from subsidence issues. Both also show a settlement curve exhibiting three linear phases not consistent with consolidation theory. No answer to this phenomenon has yet been proposed.

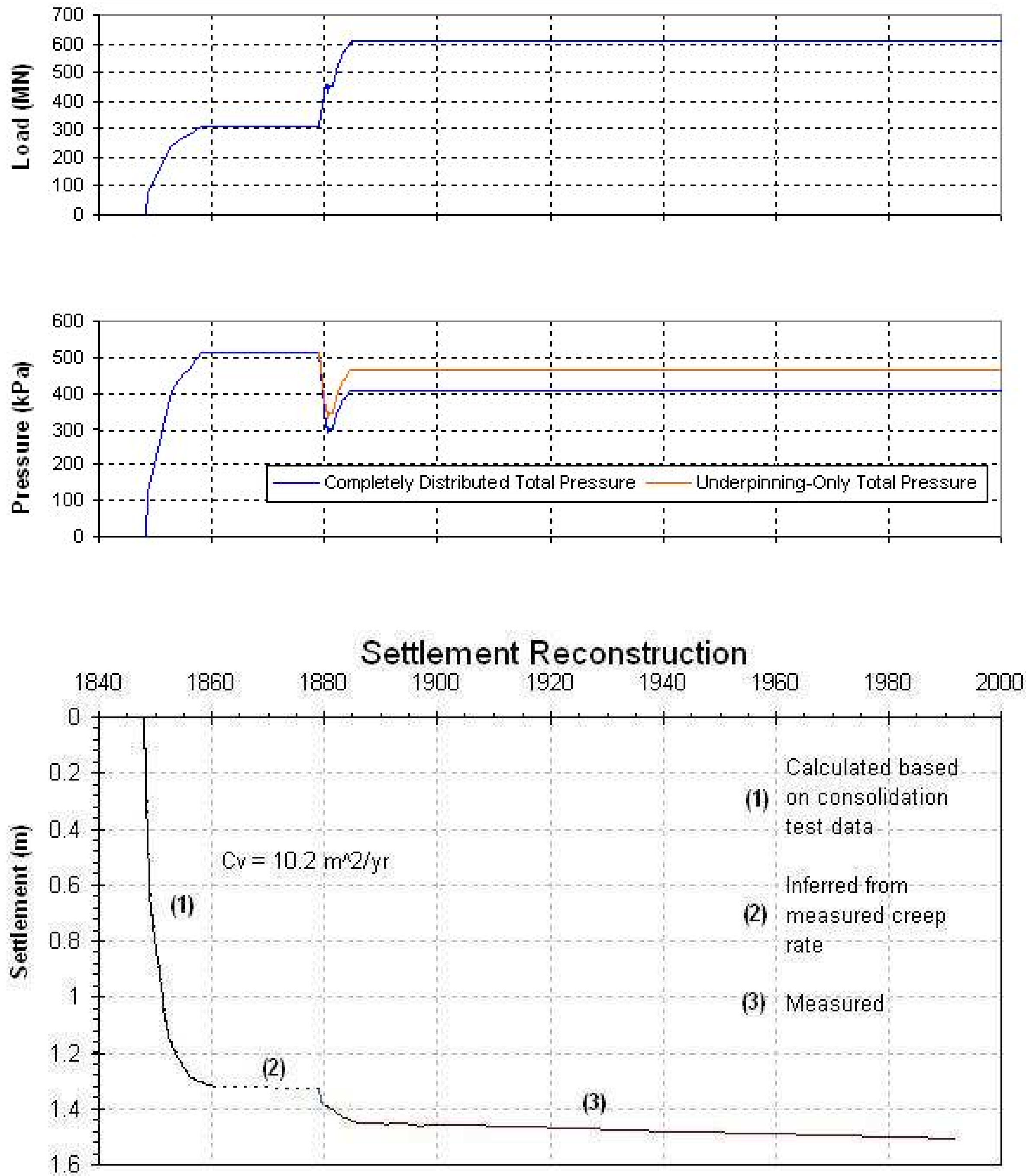

Figure 13. Reconstructed load-pressure-settlement curves vs time.

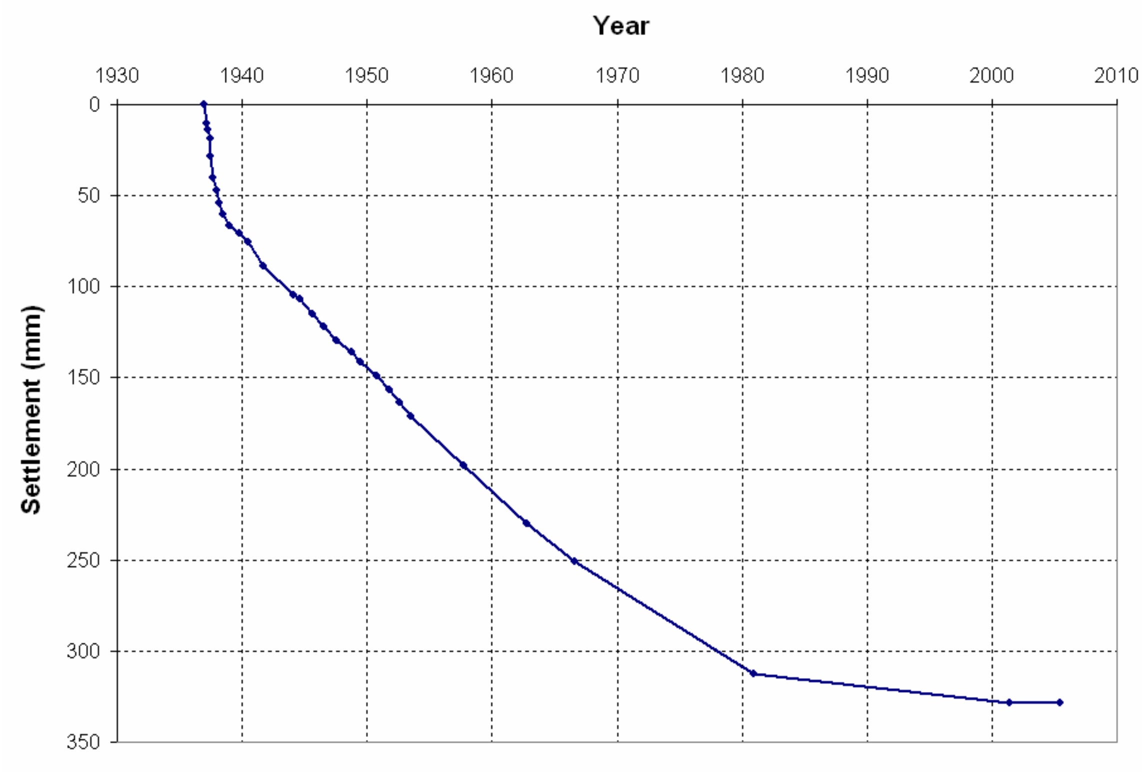

Figure 14. Settlement vs. time for San Jacinto Monument.

CONCLUSIONS

The Washington Monument is a 169.16 m high obelisk resting on a mat foundation. The construction of the first foundation started in 1848; it was a 24.38 m square mat made of gneiss blocks cemented together and organized in a pyramid. When the obelisk reached a height of 55.5 m the pressure on the soil was 513 kPa and construction was stopped because of funding problems and the Civil War. Calculations indicate that at that time the applied pressure was close to the ultimate bearing capacity of the soil and that the settlement would have reached over one meter. In 1878, construction started again by the underpinning of the existing foundation. The foundation became a 38.54 m square mat which decreased the pressure to 351 kPa; yet 52 mm of additional settlement took place during underpinning. The column was completed and the final pressure on the soil under the foundation was 465 kPa. The additional settlement from the end of underpinning to 1992 has been 119 mm; this compares favorably with the calculations based on the consolidation tests available. This case history documents the behavior of one of our important national monuments, demonstrates the value of underpinning to save a structure which was otherwise doomed to failure, and confirms the validity of the consolidation theory.

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REFERENCES

Abbot, Henry L. (1897). "Memoir of Thomas Lincoln Casey: 1831-1896." Read before the National Academy, April 21, 1897. Transcription available online at: http://books.nap.edu/html/biomems/tcasey.pdf.

Barber, Edward S. (1962). "Loading Limitations - Washington Monument Grounds." Report for National Parks Service. December 31, 1962.

Biot, M.A. (1941). "General Theory of Three-Dimensional Consolidation." Journal of Applied Physics, Vol. 12, p. 155-164.
Briaud J.-L., Nicks J., Rhee K., Stieben G. (2007). "The San Jacinto Monument Case History." Journal of Geotechnical and Geoenvironmental Engineering, 133 (11), ASCE, Reston, Virginia, USA.
Briaud J-L., Gibbens R. (1999). "Behavior or Five Large Spread Footings in Sand." Journal of Geotechnical and Environmental Engineering, 125 (9), ASCE, Reston, Virginia, USA.
Darton, N.H. (1950). "Bedrock Surface of the District of Columbia and Vicinity." Geological Survey Professional Paper 217. US Government Printing Office, Washington D.C..
Fellenius, B.H., Ochoa, M. (2009). "Discussion: San Jacinto Monument Case History." Journal of Geotechnical and Geoenvironmental Engineering, 135 (1), ASCE, Reston, Virginia, USA.
Focht J.A., Khan F. R., Gemeinhardt J. P. (1978). "Performance of One Shell Plaza Deep Mat Foundation", Journal of the Geotechnical Engineering Division, 104 (GT5), 593-608.
Holtz, R. and Kovacs, W. (1981). An Introduction to Geotechnical Engineering. Prentice-Hall: New Jersey.
Jamiolkowski, M.B. (2006). Behavior of the Leaning Tower of Pisa During and After Stabilization Works, Peck Lecture presentation.
Kulhawy, F.H. and Mayne, P.W. (1990). Manual on Estimating Soil Properties for Foundation Design. Final Report 1493-6, EL-6800. Electric Power Research Institute: Palo Alto, CA.
Mueser Rutledge Consulting Engineers. (2002). "Subsurface Investigation – Monument Grounds and Visitor Facility – Washington Monument." Report for National Parks Service, Olin Partnership, and Hartman Cox Architects. June 10, 2002.
Mueser, Rutledge, Johnston, & DeSimone Consulting Engineers. (1984). "Settlement Analysis for Washington Monument Grounds Improvement." Report for National Parks Service. February 13, 1984.
Mueser, Rutledge, Wentworth, & Johnston Consulting Engineers. (1973). "Subsurface Investigation and Foundation Study for Interpretive Facility at the Washington Monument." Report for U.S. Department of the Interior and National Parks Service. September 14, 1973.
Nicks, J.E. (2005). "A Settlement Analysis of the San Jacinto Monument." Master of Engineering Report, Texas A&M University, College Station.
NGS (National Geodetic Survey). (2009). "Meridian Stone Datasheet." The NGS Data Sheet, http://www.ngs.noaa.gov/cgi-bin/ds_desig.prl.
NPS (National Parks Service). (2009). "The Washington Monument: Tribute in Stone," Teaching with Historic Places Lesson Plans, http://www.nps.gov/history/nr/twhp/wwwlps/lessons/62wash/62wash.htm.
Olszewski, George. (1971). A History of the Washington Monument 1844 – 1968. Office of History and Historic Architecture, United States Department of the Interior. Washington D.C.: Eastern Service Center.
Parkhill, S.M. (1957). "Washington National Monument," Compressed Air Magazine, September 1957, 266-269.
Poulos H.G. and Davis E.H. (1974). Elastic Solutions for Soil and Rock Mechanics. New York: John Wiley & Sons, Inc.
Sigdel, L.P. (2007). The Eiffel Tower Case History, Master of Engineering Report, Texas A&M University, Zachry Department Of Civil Engineering, College Station, TX, USA.
Skempton, A.W. and Bjerrum, L. (1957). "A Contribution to the Settlement Analysis of Foundations on Clay." Geotechnique, Vol. 7, p. 168.
Smith B.J. (2007). The Washington Monument Case history, Master of Engineering Report, Texas A&M University, Zachry Dpt. of Civil Engineering, College Station, TX, USA.
Torres, Louis. (1985). "To the Immortal Name and Memory of George Washington / The United States Army Corps of Engineers and the Construction of the Washington Monument," Historical Division, Office of Administrative Services, Office of the Chief of Engineers. Washington D.C.: U.S. Government Printing Office.
U.S. Geologic Survey. (1950). The District of Columbia, Its Rocks, and Their Geologic History. U.S. Geologic Survey Bulletin 967-968. Washington D.C.: U.S. Government Printing Office.

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