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FOUNDATION FOR THE PALACE OF THE SOVIETS.
DESIGNING A RIGID HEAVY FOUNDATION ON COMPRESSIBLE MATERIALS THROUGH THE USE OF SOIL MECHANICS

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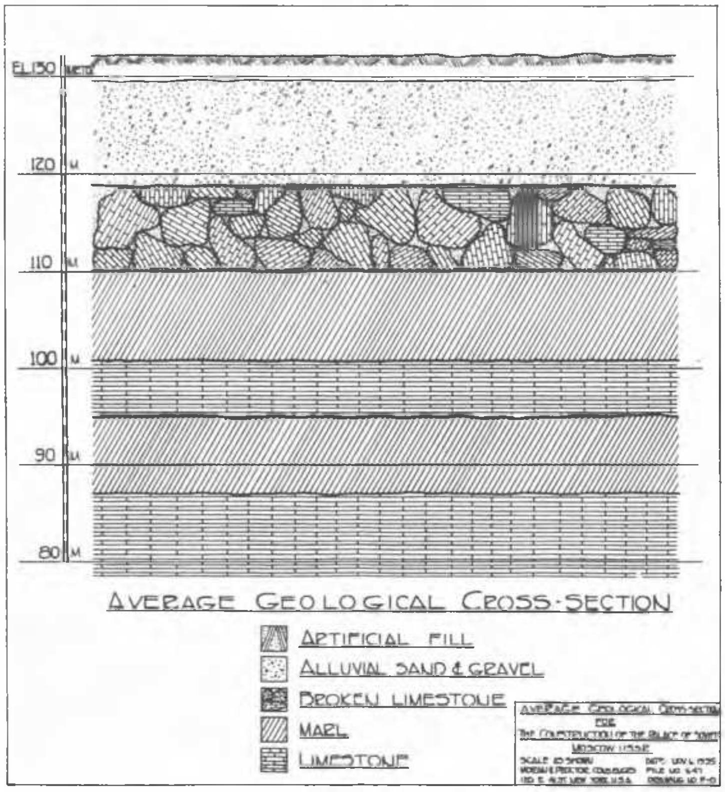
Engineering records are replete with examples of foundations where accuracy and economy of design may be accredited to the contributions of soil physics, but there is one case where the adopted design was made possible through the employment of soils mechanics. I refer to the foundations for the Palace of the Soviet at Moscow, where both the geology of the site and the type and design of the superstructure presented entirely unusual problems to the foundation design.



Situated just east of the Kremlin on the bank of the Moscow River, the site presented an average geological cross-section as shown in the figure at the left. Below average grade at elevation plus 132 meters, about 2-1/2 meters of artificial fill overlie an alluvial deposit of sand and gravel. At an average elevation of 119 this sand and gravel stratum meets a broken limestone stratum which has been shattered and laterally moved towards the river by glacial forces. This shattered limestone averages in thickness about 9 meters and is underlain by a stratum of marl averaging 9 meters thick which in turn rests on an undisturbed limestone bed averaging about 6 meters thick. Under this second limestone stratum is found another stratum of marl averaging 8 meters thick resting on a continuous bedrock limestone at average elevation plus 87 meters.

But the actual varies widely from this average geology, particularly as to the character, compressibility and thickness of the marlaceous materials between the second and third limestones and as to the thickness of the second limestone. While this intermediate limestone stratum varies from a minimum of 4 meters to a maximum of 8 1/2 meters, the underlying marlaceous materials vary from a practically incompressible marl limestone to a readily compressible clay marl, and this marlaceous stratum itself varies from 4 1/2 meters to 10 meters in thickness. Furthermore, variations in thickness, compressibility and interbedding of softer lenses were found over short horizontal distances.

The foundation problem resolved itself into a question of - first, the maximum differential settlement which the structure would safely withstand, and second--the level of bearing which would provide such required result, i.e. the depth to which the supports must be carried.

It was determined that cofferdamming provisions could be developed which would permit of open installation to the top of the second, or intermediate, limestone, but the large volume and high head of flow through the limestone indicated the probability that pneumatic installation might probably be required to reach the third limestone, and in that event that the increased cost of such installations would be very large. Therefore, efforts were made to arrive at a safe and economical design to support the tower columns on the second, or intermediate, limestone.

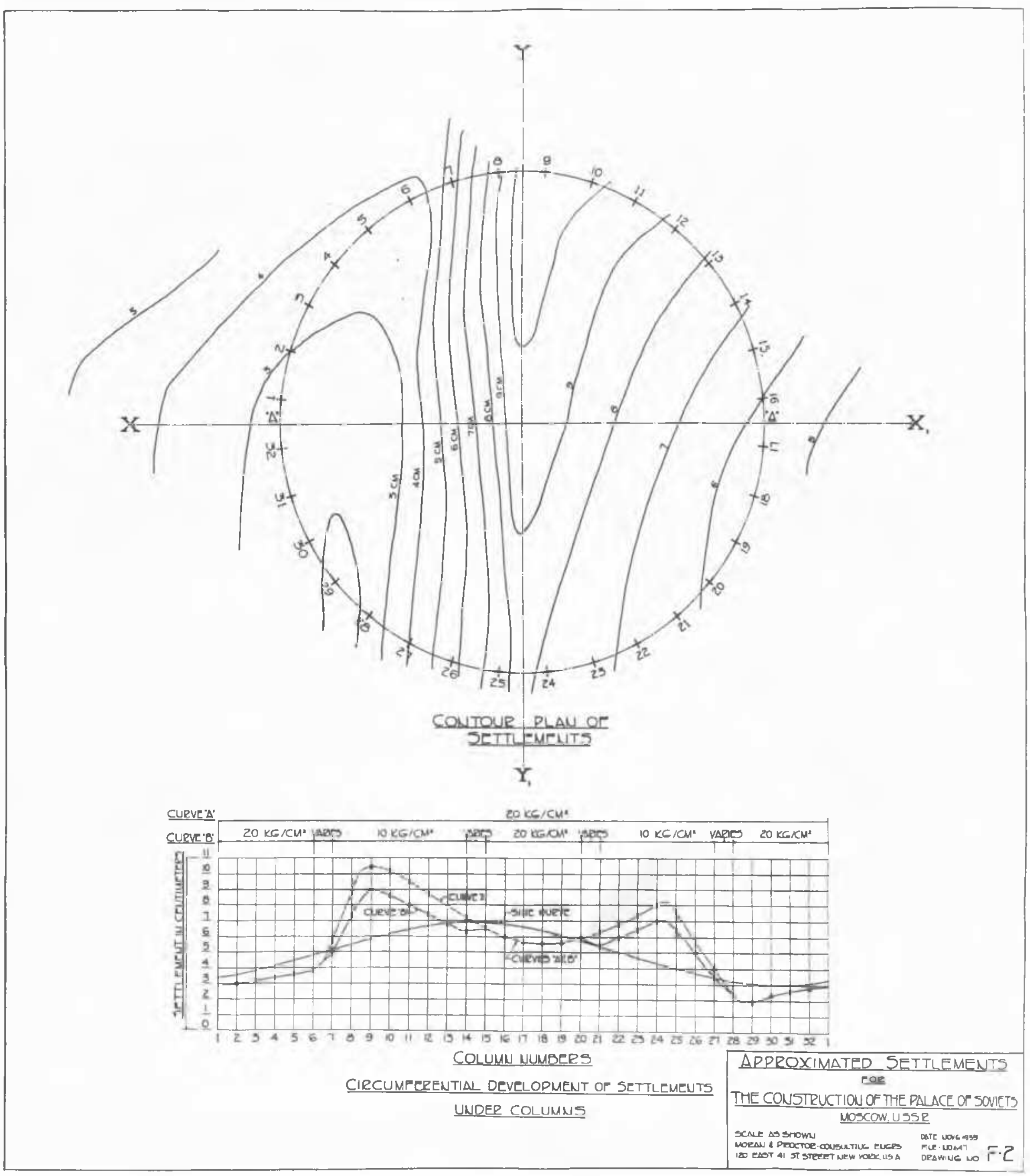
The superstructure design is unprecedented and one which lends itself to but slight differential settlements. Most simply stated the superstructure steel skeleton consists of a single set of two leg columns placed on the perimeter of the tower base, with no interior columns. The diameter of the circle of the inner column legs is 140 meters and that of the outer column legs 160 meters. These columns set in from the vertical as they rise around the hemispherical dome of a circular auditorium 140 meters in diameter and approximately 100 meters high, by 3 successive inward breaks from the vertical. At the top of the dome the columns resume verticality, so that at this level there is a large inward horizontal component of column loads which must be resisted by an annular girder acting as a circular horizontal column. Any differential settlement at this level would cause eccentric bending stresses in the annular compression girder, which unless strictly minimized are obviously inadmissible in such a design.

Therefore, any foundation design contemplating support on the intermediate limestone must provide that the foundation structure, together with the superstructure frame below the top of dome, resist and counteract inherent tendencies to differential consolidation of the underlying marls, in excess of such consolidation as would not cause the base of columns to depart from a plane.

Applying the results of laboratory consolidation tests on undisturbed samples of the marlaceous materials lying between the second and third limestones, the amount of consolidation which would result from isolated column footings supported on these materials was calculated, together with the effect on such consolidations, by variations in the intensity of loading on the upper surface of the second limestone.

Vertical components of soil stresses were calculated by the use of Boussinesq's formulae, at depths below the top of second limestone of 5, 10, 15 and 20 meters respectively. For the purpose only of determining soil stress distribution, piers were assumed circular and of diameters corresponding to gross bearing intensities of 10, 15, 20 and 30 kg/cm^2, which respectively are equal to net soil intensities (that is, increase over existing pressures within the soil) of 6.88, 11.88, 16.88 and 26.88 kg/cm^2. The center of gravity of the maximum thickness of compressible materials lies generally from 10 to 11 meters below the top of the second limestone, and the above determinations indicated that in the center of depths of the compressible zones vertical soil pressures were changed less than 20% when the bearing pressure at the top of the second limestone was decreased from 20 to 10 kg/cm^2, thus indicating that the settlements under the heavy tower would be primarily a function of the thickness of the compressible materials lying between the second and third limestone and would be affected only to a comparatively small degree by variations in the intensity of foundation pressures at the top of the second limestone.

By use of the consolidation curves for undisturbed samples, settlements under assumed isolated piers were calculated to vary from a minimum of 1.2 to a maximum of 9.3 cm, under 10 kg/cm^2 intensity; from 1.7 to 10.8 cm, under 20 kg/cm^2; and from 1.9 to 11.2 cm, under 30 kg/cm^2; with a maximum differential settlement between immediately adjacent columns of 3 1/2 cm. under a 20 kg/cm^2 intensity. Utilizing these



determinations a plan was developed showing contours of settlement which would result from isolated column footings bearing at 20 kg/cm^2, such plan being merely an interpolation of settlements arrived at by assuming that at each boring location intensities on the various subsoil strata would be those existing under isolated tower columns if the entire ring of loaded columns were moved so that one column was situated at the location of the particular boring. This drawing also gave a circumferentially developed profile of settlements plotted from the settlement contours, and a profile of vertical consolidations based on an intensity of 20 kg/cm^2 where the tendency to settlement was least, decreasing to an intensity of 10 kg/cm^2 where the tendency to settlement was greatest. Superimposed on this last profile was a sine curve of anticipated consolidation under a continuous annular reinforced concrete girder carrying the column towers, such sine curve being based on a slight anticipated tilt of the structure across a major axis between minimum and maximum marl consolidations, and such sine curve indicating the maximum consolidations permissible without departing from the required plane through the column bases and thus maintaining the annular compression girder at the top of dome in a plane.

Additional undisturbed sample borings and consolidation tests were called for to accurately determine the above discussed factors at each column preparatory to the accurate plotting of consolidation curve for isolated footings, and the departure of such curve from the sine curve would determine the extent to which a continuous concrete foundation girder must distribute certain column loads to adjacent areas of lower compressibility.

Inasmuch as the foundation depth from the column base to the top of the second limestone is 22 meters, this depth will be utilized as the depth of the annular foundation girder which will be designed to even out unequal settlements by the distribution of column loads over considerable distances.

In designing the foundation girder wall, the first problem to be solved lay in the determination of the amount of settlement which would occur if each column were supported on an isolated footing and there were no stiffening ring between such footings. By considering all column loads, except that at the point under consideration, as point concentrations, Boussinesq's formula is used to secure the vertical components of stress at the point in question due to surrounding loads. At such point, the vertical component of stress may be computed by considering the load as applied uniformly at a given intensity over the rectangular wall area, and the vertical stresses may be computed at various levels under the center of such area. Total settlements may be computed by adding the load at the column center to those resulting from surrounding loads and applying the consolidation curves and boring records as to character, amount and dispersion of compressible materials. If within the range of the loads considered, the consolidation curve is assumed as a straight line, it can be said that the total settlement is composed of two parts, i.e. that part due to the load at the point in question which will vary directly with such load and that part due to surrounding loads which is fixed in magnitude and existing in the same amount irrespective of the load at the point under consideration.

Stiffening of the superstructure will provide an adequate factor of safety to permit the assumption in the design that the girder wall is rigid. A discussion of the actual design of the wall is not pertinent to this paper. The intention has been to indicate the present day value of the science of soils mechanics, in having facilitated a design where definite consolidation factors and subsoil data were essential. Without this relatively new engineering tool, the best assumptions as to subsoil compressibility would have been inadequate and the only alternative would have been a foundation supported on the deep third limestone which would have involved a very considerable increase in cost, in time and in hazard of installation.

No. N-2

FOUNDATIONS OF THE NEW TELEPHONE BUILDING, ALBANY, N. Y.

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When the new building for the New York Telephone Company was erected in Albany, N.Y. in 1929, the foundation problem was solved in a manner differing radically from usual practice for such work. At that time the writer published a paper (See bibliography item 1) which briefly outlined the foundation conditions and the method of solution. Because the unusual soil formation and the novel type of building foundation adopted have been of considerable interest to engineers practising along these lines, the facts leading up to this design are presented in this paper.

The soil formation on Capitol Hill, upon which this structure is built, has long been a source of foundation problems and the settlements of structures in this locality have taxed the ingenuity of engineers and architects engaged in their planning. Numerous borings taken from time to time for various structures indicate that the character of the soil is uniform throughout this area and consists of thin interbedded layers of Clay and sand, the clay varves varying from a few millimeters to several centimeters in thickness, and these separated by thin layers of fine sand, in most instances not more than a millimeter in thickness. The clay portions of the soil contain varying degrees of moisture as shown in Fig. 1, which is a diagram of an analysis of the soil prepared by Dr. A. Casagrande (2). The thin layers of sand which may be observed in any excavation in this material, serve as excellent drainage channels, and water freely drains from the exposed faces.

One of the earliest recorded studies of the action of this soil under load was undertaken by W. J. McAlpine (3) (4) at the time of the construction of the New York State Capitol Building. As a result of these tests McAlpine concluded that the soil would fail at less than six tons per sq. ft. and that a safe value should not exceed two tons per sq. ft. In the final design of the foundations for the