Restoration of an Old Building in San Francisco

La Restauration d'une Vieux Bâtiment à San Francisco

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SYNOPSIS A search for a building to house the Far West Laboratory Educational Facilities found an old 6-story reinforced concrete building about 2 kilometers south of the City of San Francisco civic center. Original plans of the building disclosed that it was supported on timber piles of unknown lengths in groups of 11 to 14 under each column. The undulating floors indicated that the building had under gone significant settlement. Updating the building including additional seismic resistant reinforcement required that average loading on supporting piles would increase from 13,500 kg to 18,000 kg per pile. Subsurface exploration, laboratory testing and 2 pile load tests were performed. Excavations for the pile load tests disclosed that the piles were below the water table and exhibited no signs of decay. Settlement and pile load test studies disclosed that the building load plus downdrag forces exceeded the supporting capacity of the piles and that the building was settling at about the rate of consolidation of the underlying "recent Bay Mud". However, relative to downdrag forces the proposed additional building loads were deemed tolerable with the predicted future settlements being in the order of 0.03 to 0.06m. over the next 20 years.

INTRODUCTION

In 1968 a consortium of universities in the western United States of America formed a joint powers agreement to develop an educational laboratory. The resulting organization, the Far West Laboratory Educational Facilities, began a search to find a building to house their offices. The structure chosen was an old, 6-storied, reinforced concrete warehouse about 2 kilometers south of the civic center of the City of San Francisco, California.

The subject building is approximately 11 kilometers east of the San Andreas rift zone, an area where many earthquake epicenters have been located, including that of the great earthquake of 1906. A search of old maps and files on the area indicates that at one time the site was probably a part of a narrow estuary which traversed the site in a north-south direction. This arm connected with a small bay, which was part of the larger San Francisco Bay.

As the City of San Francisco developed, between 1847 and 1873, ancient sand dunes were leveled and excess eolian sands were used to fill in the marsh and shallow bay areas south of the downtown area, possibly including the subject site. Further, it is believed that the site became part of a dumping ground for rubble generated during the clean-up operations after the earthquake and fire of 1906.

The building was constructed in 1927. The old warehouse was approximately 59 meters wide by 75 meters long, with the long side extending in the north-south direction. Original plans of the building disclosed that it was supported on timber piles of unknown length in groups of 11 to 14 supporting each column, thus it is probable that the piles were designed for total loads of 23 metric tons with at least 13.5 tons dead load. The existing interior columns were designed for total loads in the order of 227 tons (135 tons dead load).

Building codes used in 1969 had more stringent requirements for seismic resistant design than had been in practice in 1927. This meant that considerable lateral bracing would be required to render the old building usable for the proposed project. This addition of dead load to the building meant an increase of total loads on foundation system by about 50 percent. Because of the needed additional loading, the unknown condition and support capacity of the piles and the question of the current settlement rate of the building, an extensive investigative and pile load test program was developed.

SUBSURFACE EXPLORATION

A total of 5 borings were advanced adjacent to the building to a maximum depth of 40 meters. Locations of the borings are shown on the site plan, Figure 1. Drilling of the borings was accomplished using a truck-mounted Failing 1500 rotary wash drilling rig. Bentonite drilling mud was used to stabilize the holes and return drill cuttings. Samples were generally collected at 1.5 to 3m intervals.

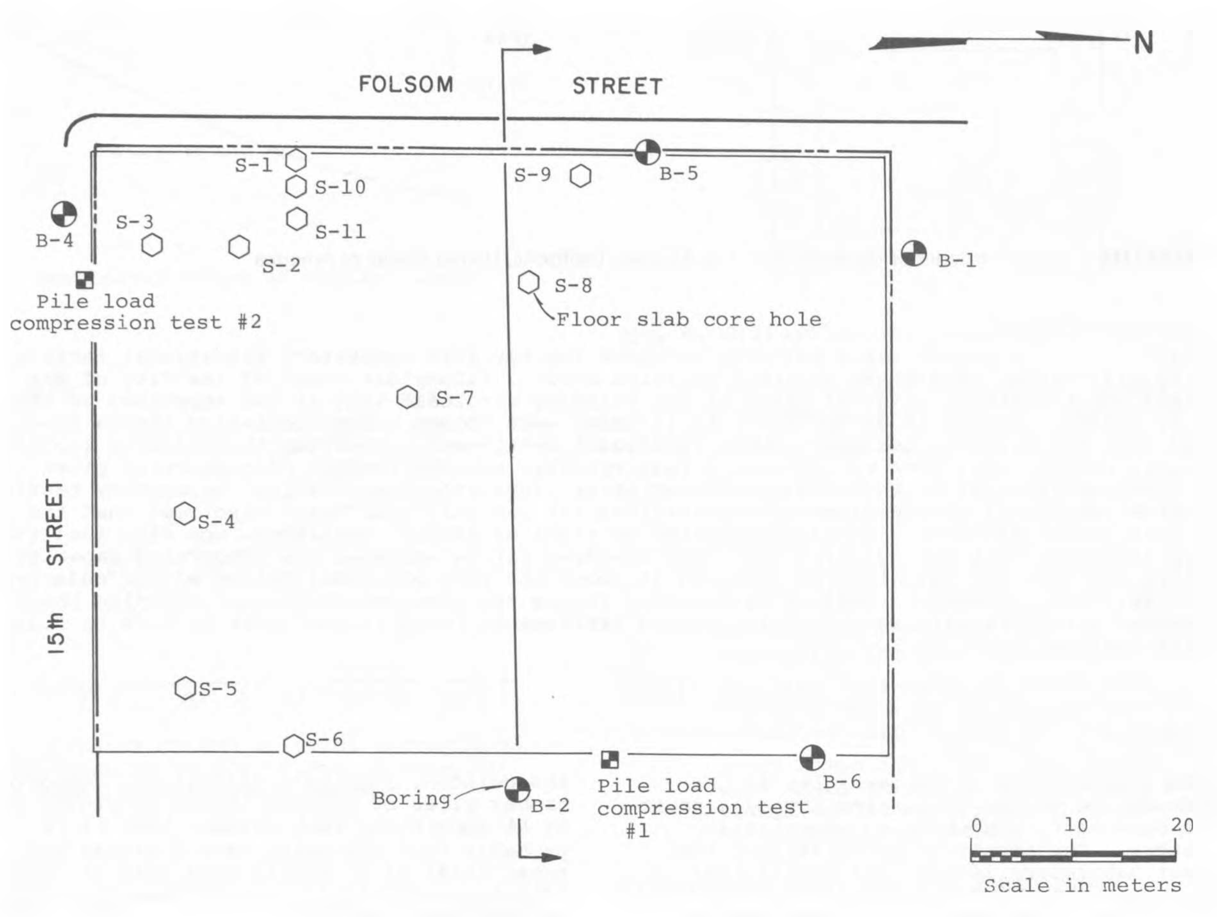

FIGURE 1: BUILDING SITE PLAN

Most samples were taken with a 50.8mm O.D. split spoon sample driven into undisturbed ground with a 63.5 kilogram hammer dropping freely a distance of 0.76m. The blow count (Standard Penetration Resistance, N) was recorded and was used as an evaluation of the relative density of the subsurface material. In addition to the Standard Penetration samples collected, relatively undisturbed 76mm O.D. thin-wall tube samples were taken in cohesive soils by pushing the sample tube with a hydraulic ram on the drilling rig.

A stand-pipe type piezometer was installed in one of the borings to develop groundwater flucuation data. Groundwater was generally found to be about 3m below the ground surface, and flucuated less than 0.3m over the 1½ years observed.

Eleven core holes were cut through the concrete floor slab on the ground floor of the building for the purpose of determining if fill had settled away from the floor slab.

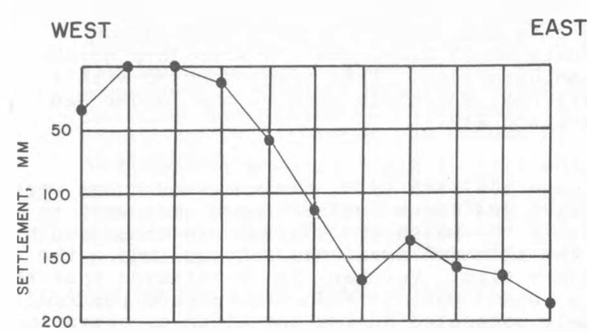

FIGURE 2: SECOND FLOOR PROFILE

In general, larger voids were noted beneath the floor slab beneath the western portion of the building. Measured voids beneath the slab ranged from 7mm to 65mm. In addition, a void space less than 10mm was observed beneath the center of the pile cap at the westernmost pile load test site. Rutherford and Chekene Structural Engineers, the firm retained to provide structural redesign of the building, took water level surveys on the first, second, third and sixth floors. Data from this material showed maximum differential settlement across the building from east to west was about 230mm and the average differential settlement was about 165mm. An east-west profile taken on the second floor is presented in Figure 2. City survey monuments had been established in the street in front of the building in 1953 and data from those monuments suggests that settlement of the site is still occurring at a rate less than 0.5mm per year.

LABORATORY TESTING

Laboratory testing consisted of the conventional determination of natural water content for all samples above the water table and all clay samples, Shelby tube samples (thin-wall tube sampler) were also tested for shear strength using the unconfined compression test and the Torvane shear test (hand-held vane shear). Dry unit weights were determined for all undisturbed samples. In addition, consolidation tests were performed on samples of the recent soft clay (Bay Mud) and older stiff clays.

SUBSURFACE CONDITIONS

The general stratigraphy at the site was 5 to 6 m of fill underlain by up to 9.8m of uniform, fine to medium, silty sand which is in turn underlain by recent Bay Mud.

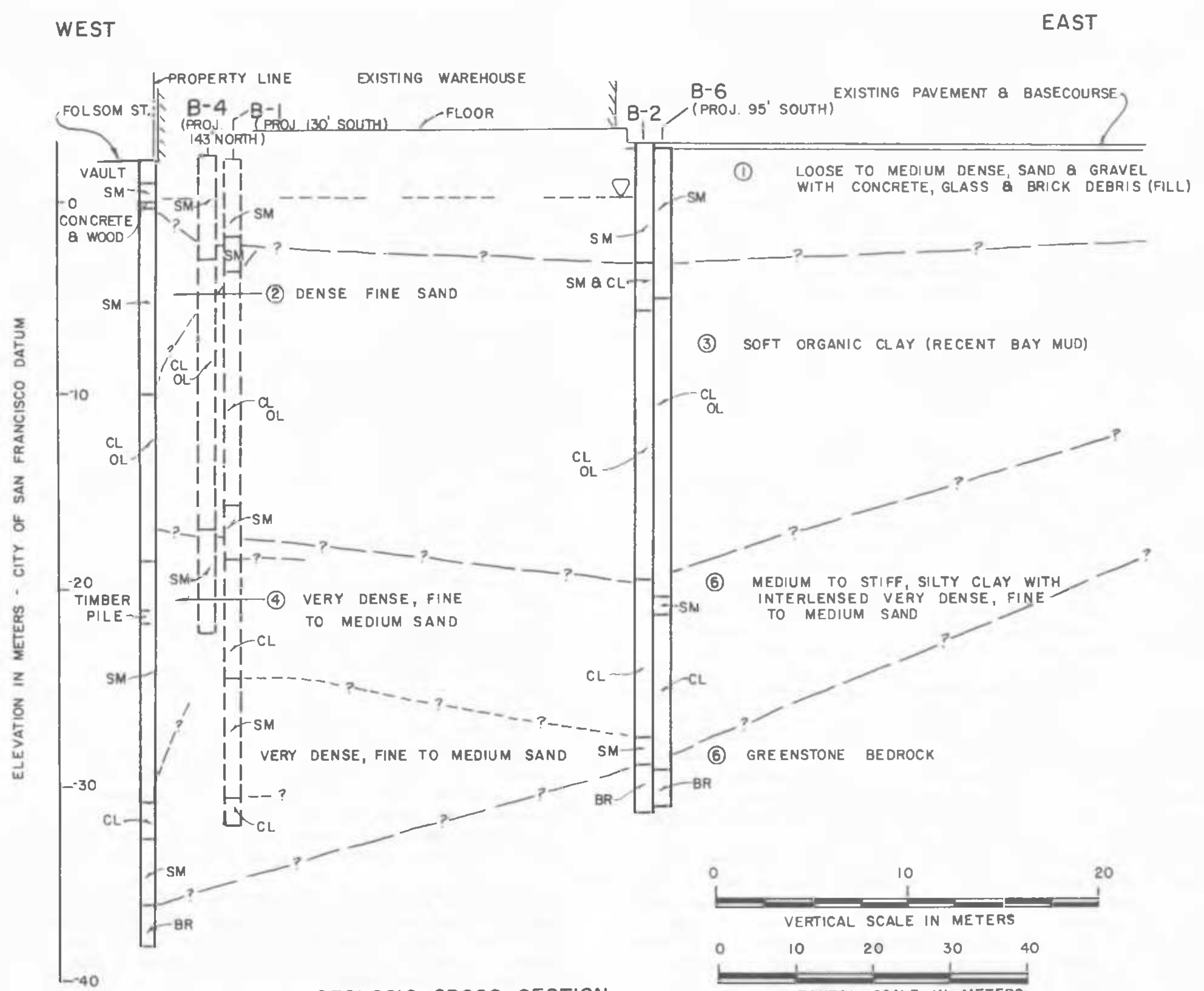

FIGURE 3: GEOLOGIC CROSS-SECTION

Underlying the Bay Mud was found an interbedded or interlensed dense sand and stiff clay which overlies serpentinized greenstone bedrock. Beneath the eastern portion of the building, bedrock was found at a depth of 31.7m and beneath the western portion of the building, bedrock was found at a depth of 38.3m. A geologic cross-section shown on Figure 3 illustrates the generalized soil conditions in the east-west direction through the center of the building. Strata were more uniform in the north-south direction.

The fill overlying the site is principally loose, fine to coarse sand with some gravel and varying amounts of concrete and glass rubble. The fill is identified as Layer 1 on Figure 3. A layer of uniform fine sand (Layer 2) was found at the base of the fill having a thickness up to 9.8m. This material may have been wind-deposited, being blown in from sand dunes to the west, or it may have been placed by man, as suggested earlier in this paper, as part of the development of downtown San Francisco. Standard Penetration resistance in the fill ranged between about 2 to 15 blows per foot, and that of the uniform fine sand ranged between about 20 and 40 blows per foot.

The recent Bay Mud (Layer 3) found at depths varying between 4 and 12m below the ground surface, was a gray, very soft to soft, organic, medium to highly plastic clay with traces of peat and shells and occasional thin seams of fine sand. The Bay Mud was found to be thinnest (8.5m) beneath the west central portion of the building and thickest (15m) beneath the back central portion of the existing building. Across the front of the building the thickness of the Bay Mud varied from 8.5 to 13.7m. "N" values in the soft organic clay ranged between 0 (sampler penetrated under the weight of the rods) and 5 blows per foot. The shear strengths ranged between 0.4 to 0.63 kilograms per square centimeter. Remolded shear strength of the soft clay was in the order of 0.1 to 0.2 kilograms per square centimeter indicating a sensitivity between 3 and 4. The consolidation tests indicated that the soft clay was nearly normally consolidated under the existing overburden pressure. It is interesting to note that the compression index in the consolidation tests was very similar to values obtained from samples of Bay Mud from other thick deposits found throughout the San Francisco Bay.

The sand is thickest beneath the front center of the building and wedges out beneath the back of the building. Standard penetration values of the sand were over 100 in all cases. Immediately underlying the dense sand and recent Bay Mud in one boring was a gray, medium to stiff silty clay (Layer 5) which was identified as old Bay Clay, ranging in thickness to a maximum of 9m. Interbedded or interlensed with the stiff clay was found very dense fine to medium sand in lenses up to 5m in thickness. Standard penetration values in the stiff clay ranged between 5 and 14 blows per foot, and moisture content varied between 40 and 70 percent, averaging about 55 percent. Measured shear strengths ranged up to 0.6 to 2.0 kilograms per square centimeter.

Standard penetration values of the dense to very dense sand interbedded with the clay ranged from 30 to over 100 blows per foot. A consolidation test performed on a sample of the stiff clay indicated that the clay had been overconsolidated by about 2 kilograms per square centimeter above existing overburden pressure.

Moderately hard, fractured greenstone (Layer 6) was encountered between depths of 31.7m and 38.3m. The bedrock in the area consists of the Franciscan Formation which is of the upper Jurassic to Cretaceous age. At the site, the greenstone was partially altered to serpentine.

PILE LOAD TESTS

Two pile load tests were performed beneath areas of the building known to have differing subsoil conditions, in general the difference between the east and west sides of the building. The purpose of the testing program was to establish the ultimate capacity of the test piles and, if possible, to determine the percentages of the pile capacity developed by skin friction and by end bearing. Excavations for the pile load tests disclosed that the piles were below the water table and exhibited no signs of decay.

To estimate the frictional capacity of the test piles, each test load increment was applied for 30 minutes and then recycled to zero load for 15 minutes. Loads were applied in 9 to 13.6 metric ton increments. Pile Load Test No. 1 on the east side of the building developed failure at 90.7 ton and Pile Load Test No. 2 developed failure at 95.2 ton. Each test was concluded by application of approximately 2 times the proposed design load for a period of 7 to 10 hours. In both cases, significant settlement was not observed under this load after 3 minutes. A schematic of the typical pile load test details is shown on Figure 4. Although an attempt was made to distinguish between load capacity developed through skin friction and end bearing by cycling the loads, the results of the efforts were inconclusive.

No records were available at the time of the investigation to indicate the length of existing piles. It was planned as part of the investigation program to pull 2 piles tested to determine their lengths. The cost to pull the piles was excessive relative to the expected benefits, and that portion of the investigative program was eliminated. In Boring B-5 a pile was encountered between depths 22.8 and 23.6m. At this depth, the pile was 3m into the dense sand (Layer 4). This was an indication that at least some of the piles were driven through the Bay Mud and into the more competent underlying sediments.

Assuming that the piles would have penetrated through the Bay Mud, the pile tested in Pile Load Test No. 1 would penetrate into the stiff clay at the back of the building and the pile tested in Pile Load Test No. 2 would penetrate into the dense sand at the front of the building.

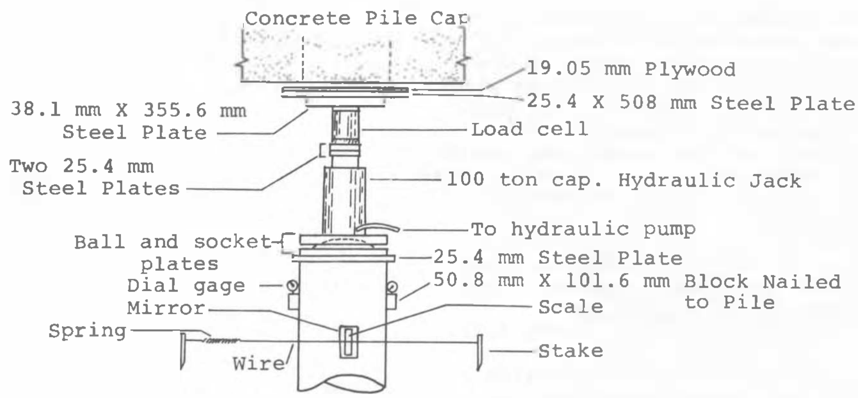

FIGURE 4: TYPICAL PILE LOAD TEST DETAIL

Hereinafter, pile conditions will be referred to as stiff clay support, or dense sand support. These two conditions will represent the extreme foundation support conditions at the site for the purpose of the following discussion. However, since the approximate size of only a few piles was known, it is possible that other piles would be somewhat smaller and shorter and would have smaller load carrying capacity. Other areas across the building were assumed to be represented by conditions somewhere between the conditions represented by the 2 pile load test sites.

The pile tested in Test No. 1, failed at a load of 90.7 ton. An evaluation of this test was made using maximum adhesion values of 0.375 kilograms per square centimeter for Bay Mud and 0.7 kilograms per square centimeter in the stiff clay. These values were selected using laboratory shear strength data as indices. The computed length of the pile tested using the above adhesion values would be about 28m below the pile cap, or, 7.6m into the stiff clay to support an ultimate load of 90.7 ton. This is a reasonable length for a pile having an average butt diameter of 380mm. Ignoring support by the relatively small amount of fill, the portion of the load carried by the Bay Mud would be 53.5 ton and that carried by the stiff clay would be 37.2 ton.

A similar evaluation of the pile tested in Test No. 2 which failed at 95.2 ton, was made assuming an angle of internal friction (ø) for the dense sand of 35 degrees. The load on this pile, if it penetrates 3m into the dense sand would be 23.6 ton end bearing plus 25.4 ton skin friction in the dense sand and 46.3 ton in the Bay Mud. The difference in the computed support divided by the Bay Mud in the two tests is due to the difference in butt diameter of the two piles tested and the thickness of the Bay Mud.

Continuing consolidation of the Bay Mud under the weight of the fill after construction of the building would produce downdrag forces in the upper portion of the pile. The settlement would cause most of the soil stresses in the Bay Mud to become negative. The computed downdrag loads would then be 53.5 ton for the pile in stiff clay and 46.3 ton for the pile in the dense sand.

This does not include the possible additional load of fill supported directly or indirectly by the pile cap. The structure load which is in the order of 13.5 ton per pile plus the downdrag forces exceed the support available in the stiff clay. Thus, it is likely that piles would tend to settle at the same rate as the fill.

The same would be true for a pile founded in dense sand; however, the support provided by the dense sand is somewhat greater than the support provided by the stiff clay. This would be particularly true if piles were driven farther into the dense sand assumed for the above evaluation, i.e. piles longer than 21m. Because the support capacity available in the dense sand was likely greater in proportion to the downdrag forces from the Bay Mud, it may be safely assumed that the rate of settlement of a pile founded in dense sand would probably be somewhat less than that of the fill. It follows that with the pile settling slower than the site fill, that voids would develop beneath the pile caps, grade beams and floor slabs. This was substantiated by the investigation of voids beneath the floor slabs. In addition, the settlement pattern across the building is consistent with piles having tips founded in dense sand in the west and stiff clay on the east.

CONCLUSIONS

The construction of the required lateral bracing and other necessary structural additions to the building were found to increase the real loads in the building by about 50 percent or to a total dead plus live load of about 20 ton per pile. Under the short term load condition, the increased load would be reflected by reduction in the value of negative adhesion in the Bay Mud. The maximum downdrag would ultimately develop with further settlement of the ground surface with time. Generally, it was concluded that the performance of the piles would be controlled by downdrag forces and would settle at approximately the same rate as the site fill. The rate of settlement might increase slightly due to additional loads, but with time, the rate of settlement would tend to return to the present rate or less.

It was thus concluded that it would be feasible to increase the structure load as proposed. However, it was recommended that the structure be designed to be capable of tolerating settlements of 30 to 60mm in the next 20 years. It was believed that this settlement would be occurring as differential settlement with the east portion of the building settling relative to the west.

ACKNOWLEDGEMENTS

The writer wishes to thank Dr. John K. Hemphill and Mr. Paul Spain with the Far West Laboratory for Educational Research and Development, for their permission to utilize data collected for the project in preparation of this paper. In addition, the writer wishes to thank Dr. Ron Sally with Shannon and Wilson and Mr. Hal Davis with Rutherford and Chekene for their permission to utilize data collected in part by the author during the work performed by the two consulting firms.