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Foundation Heave at ASCO II Nuclear Power Plant

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SYNOPSIS Excavation of 75m of claystone at Asco Unit II resulted in foundation heave in excess of anticipated elastic rebound. The heave was found to be caused by expansive clay minerals in the rock swelling in the presence of water. Upper-bound estimates of future heave were made based on past trends, and structures analyzed to demonstrate their capacity to resist heave-caused deformations. The primary factor in controlling heave is the amount of water available to the bedrock.

INTRODUCTION

The Asco Nuclear Power Station is located in eastern Spain, adjacent to the Ebro River near the town of Asco. Exploration conducted during 1972 and 1973 for Units I and II revealed the site consisted of clayey limestone and calcareous claystone (marl) bedrock overlain by gravel and silt. Excavation for Unit II in 1974 required removing a large ridge of marl to a maximum depth of 75m. This resulted in the removal of 1,700,000m^3 of rock and overburden.

The potential for foundation rebound at Unit II was recognized during the site investigation phase. Experience on other large projects with rebound under similar geological conditions indicated that reloading effectively stopped rebound and even caused some settlement (Lane, 1953; Bara, 1967; Chang, 1970). Similarly, the rebound at Asco was expected to be elastic, of short duration, and quite small in magnitude. The investigation also disclosed the potential for rapid deterioration of the claystone bedrock when exposed to air and water. Accordingly, exposed bedrock surfaces at the foundation level were protected by a concrete cover placed immediately after excavations and provisions were made to control surface drainage.

Upward movement of the foundation was first observed in early 1975, and attained a velocity of 9.5 mm/month by late 1975 with a total heave of 4cm. While this amount of heave was consistent with the anticipated elastic rebound, the continued rate of heave after excavation led to a detailed investigation of the cause of the heave and of effects on structures. The investigation included analysis of past and future heave trends, mineralogical and swell tests on the marl, ground water studies, and computer analysis of structures to assess their capacity to resist postulated heave effects.

SITE AND FOUNDATION CONDITIONS

The Asco site is located on a broad terrace and bench within a bend of the Rio Ebro midway between the towns of Asco and Flix in Tarragona Province, Spain. Exploration for Unit II began in mid-1973, and excavation commenced in 1974. Topography at the Unit II site was dominated by an east-west trending nose of sedimentary rock carved by erosion action of the Rio Ebro. The maximum elevation of the top of the nose was greater than 100m. Since it was desired to maintain plant grade at elevation 50m as at the adjacent Unit I, the large ridge of marl 250m long, 150m wide, and a maximum of 75m high was excavated.

The general arrangement of the Unit II structures is shown in Figure 1. The major buildings include the containment, fuel, auxiliary, control, and turbine-generator structures. All structures are of massive, reinforced concrete with foundations ranging from 5 to 20m below final plant grade.

Exploration at nearby Unit I had revealed that the Asco site consisted of well stratified, essentially flat-lying clayey limestone, clayey calcareous sandstone and calcareous claystone (the marl sequence) overlain by a thick sequence of gravel and silt deposits. There was no evidence of faulting, folding or other tectonic activity in the immediate site area. Similar conditions were encountered in the eastern portion of the Unit II site in the vicinity of the turbine building. However, the alluvial deposits became increasingly thinner in a westerly direction and material excavated in the vicinity of the other major Unit II structures consisted primarily of the marl.

The rock at Unit II is predominantly a hard, well cemented and indurated, red to reddish brown claystone interbedded with softer claystone and mudstone strata and containing veins of gypsum or anhydrite. Where unweathered

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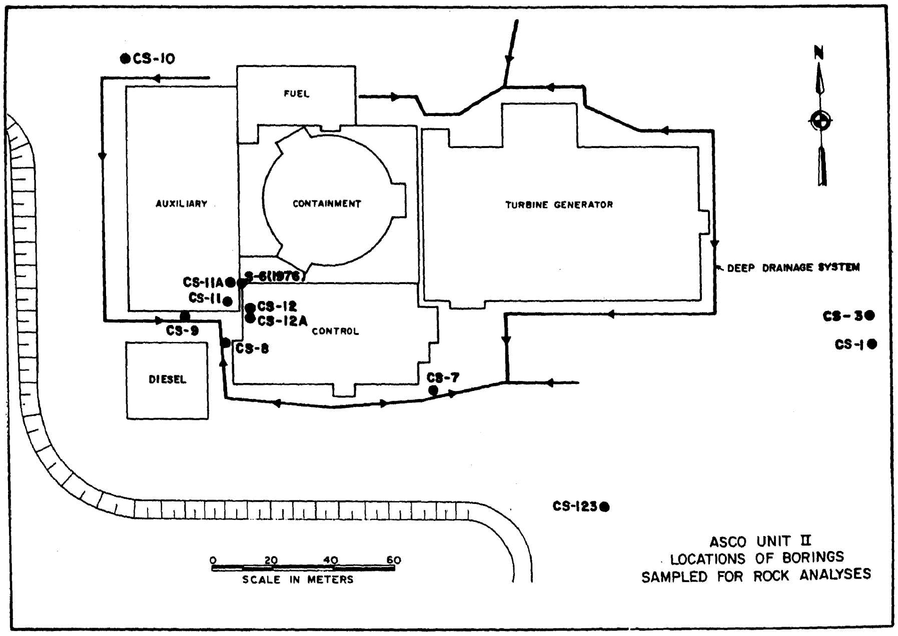

Fig. 1 Unit II Site Plan

or unfractured, the claystone rock strengths are frequently greater than concrete. These harder zones are gypsiferous claystone which is well cemented with a calcareous cement. The weaker zones are softer, less cemented materials quite subject to weathering. Dessication causes fracturing, cracking, and spalling. There was no evidence that the bedrock was subject to large residual locked-in stresses. Ground water was not encountered above elevation 23m.

HISTORY OF HEAVE

Site excavation at Unit II took place during the eight-month period from February to October, 1974. Over 1.7 million cubic meters of rock were removed from the narrow ridge occupying most of the area. The depth of excavation varied widely over the site with a maximum of 75 meters removed over the center of the auxiliary building. The resulting stress relief ranged from 70 to 170 T/m^2. At the completion of excavation, the exposed rock surfaces of cut slopes were covered with a lean concrete base.

The first indications of any abnormal foundation activity were noted in early March, 1975. At that time a system of cracks was observed to be developing in the protective mat, particularly at the auxiliary building. Cracks in the gunite and lean concrete 1-2cm wide with vertical offsets of 6-8mm toward the deep excavation were noted. A resurvey of benchmarks set in November, 1974 about six weeks after excavation was completed showed 10mm of upward movement of the mat had occurred over a four month period.

By July of 1975, leveling surveys of the bedrock indicated a maximum rebound of 4cm. This was in general agreement with the magnitude of elastic rebound calculated for the site. However, elastic rebound usually occurs during the excavation process and ceases shortly after completion. Even if unusual rock characteristics were retarding the rebound, continued movement of the magnitude recorded was considered excessive. Accordingly, it was presumed that elastic rebound was only a partial contributor to the movement and that other, probably shallow, phenomena existed. The shallow nature of the vertical movement was verified by observation of points at depths of 3, 5, and 8m below the bottom of the excavation at the auxiliary building which showed progressively less movement with depth. In addition, there appeared to be distinct correlation between periods of heavy rainfall and accelerated upward movement. Based on this evidence, the movement beyond anticipated elastic rebound was visualized as being primarily due to swelling as a result of water gaining access to the foundation rock along fractures developed through stress relief. The possible cause of the swell in the presence of water was hypothesized to be either expansion due to hydration of anhydrite disseminated in the marl or the presence of expansive clays in the marl sequence.

Recognizing the potential for substantial amounts of rebound, construction continued but with precautions to minimize the effects on the major buildings. These precautions included overexcavation of weathered or fractured rock below the foundations and prompt reloading with a mat or lean concrete, construction of foundation mats in panels and walls and slabs with temporary gaps to minimize the effects of differential heave, and stringent precautions

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against wetting of bedrock in the building site areas by water from any source.

In order to monitor the upward movement of the site, over 200 survey markers were installed in and around the major powerblock structures at Unit II. Frequent precise measurements (to a fraction of a millimeter) were made, beginning in late 1974, to record the rate of movement and total amount of displacement of the rock and foundations. These points or their replacements have been monitored to the present time, providing an excellent record of the ground movement.

The results of the survey observations on foundations and bedrock surfaces are summarized in Figures 2a - 2h. These figures are isopach contour plots of average heave velocities at Unit II for approximately the same 3-month period each year from 1975 to 1982. The darker areas delineate heave rates greater than 1.0 mm/month, while the lighter areas show heave rates less than 1.0 mm/month. From the beginning of July to the end of October, 1975, heave had attained average velocities as high as 9.5 mm/month with local, total heave up to 2.85cm (Figure 2a). This early movement was mainly concentrated at the southern end of the auxiliary building and the southeast corner of the control building. As foundation loading continued during 1976 and 1977, the heave velocities progressively decreased (Figures 2b and 2c). By early 1978, when loading of the control building was complete, the average velocities had decreased to less than 1.0 mm/month (Figure 2d). The contour plots have since indicated a generally decreasing area subject to heave, with isolated residual heave values of less than 0.5 mm/month remaining in the control building area (Figures 2e - 2h).

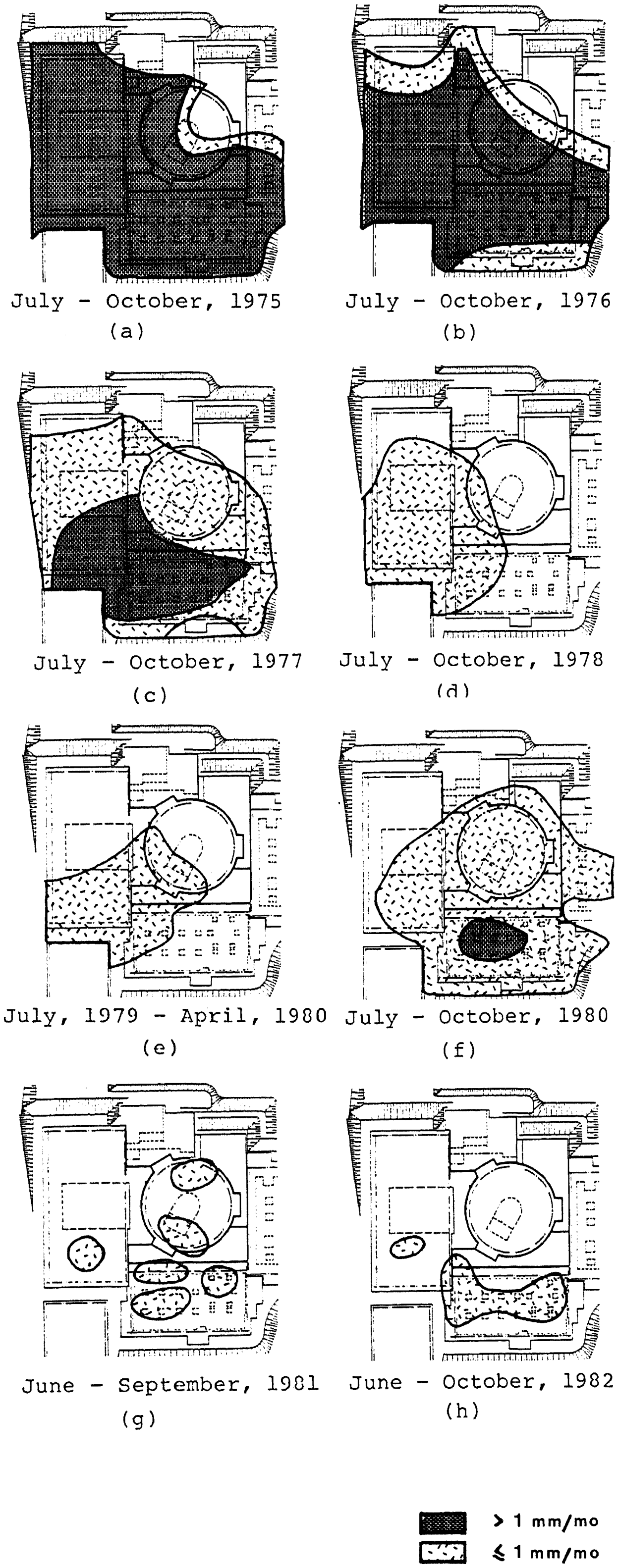

Fig. 2 Unit II Heave Rates

CHARACTERISTICS OF THE ASCO II MARL

Although construction proceeded and heave rates continued to decrease in response to loading, the persistence of heave beyond expected elastic rebound limits prompted a geotechnical investigation to identify the mechanism causing heave. Physical, chemical, and mineralogical properties of the marl and their relationship to the swelling phenomenon were studied to aid in the determination of the long-term potential for expansion. The studies centered on two probable heave-causing mechanisms: expansion of anhydrite disseminated in the marl, and expansion of clay disseminated in the marl.

Various laboratory tests were conducted to determine the mineralogy of the marl. These included x-ray diffraction analysis to determine the amount of expansive clay (smectite) present, and petrographic study of thin sections to determine relative amounts of anhydrite and gypsum in the marl and vein material. Inter-layer water content and cation exchange capacity tests were also performed.

The marl was found to be composed of calcium carbonate ranging from 40 to 70 percent in most samples, and locally up to 95 percent.

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Gypsum and anhydrite in the marl occur in amounts up to 33 and 20 percent, respectively, with the average for these two minerals being 11 and 5 percent. The veins in the marl are composed almost entirely of gypsum with minor amounts of localized anhydrite. The clay minerals present range from 10 to 30 percent of the total sample, and consist of smectite (primarily montmorillonite), illite, palygorskite, and chlorite. The average smectite content is 3 percent. Interlayer water measurements of powdered rock samples indicated that clay minerals in the samples contained in average of 2.7 percent interlayer water, of which 2.0 percent water is held by smectite. Cation exchange capacity tests indicate that 1 to 2 percent sodium cations are present in the smectite's exchange positions.

Laboratory swell tests were performed on samples of marl from foundation areas of high and low heave. The objective of the tests was to measure swell potential of the rock under simulated field conditions of lateral confinement, incremental loading, and access to water. Water with various ions present was used to observe the effect on the amount and rate of swell. Tests were conducted using specially-constructed oedometers (Figure 3). The principal differences between this oedometer and a standard consolidometer are

(a) All internal components are made of plexiglass to eliminate corrosion due to high concentrations of NaCl and SO_4 in the testing water.

(b) Samples 3.8cm square are cut from 1.3cm thick slices of rock core. These samples are immediately cast with plaster of Paris into 7.5cm diameter PVC rings for testing.

(c) Oedometers are sealed with a plastic film to prevent evaporation.

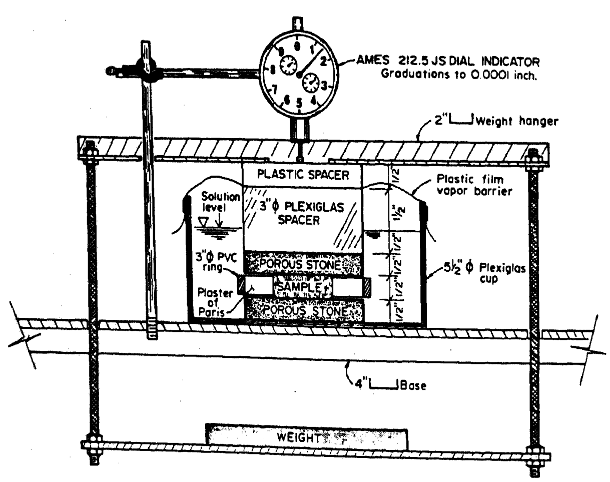

Fig. 3 Oedometer Used for Swell Tests

The results of the swell tests indicated the following:

1. Loads of between 2 and 3 kg/cm^2 precluded swelling after the initial swelling was o

2. Removal of access to water halted the swe completely. The addition of water caused abrupt and immediate resumption of swelli Expansion continued long after test specimens had become saturated.

3. There was no significant correlation betw swell and location in high or low heave areas, or between swell and various catio and anions present in the water.

4. Long-term tests on solid anhydrite did no result in any measurable swell.

HYDROGEOLOGIC STUDIES

A number of water control measures have been implemented at the Asco site in recognition o the importance of the role of ground water in the heave phenomenon. The primary water cont systems are

(a) Surface drainage and impermeabilizatio

Rainfall infiltration is eliminated as much as possible by paving yard areas and guniting slopes. Yard drains conve surface water from all sources away fr the structures.

(b) Deep drainage system.

Deep drainage systems consisting of buried concrete pipe and collector well surround both Units I and II. The deep drainage system at Unit II maintains ground water levels near elevation 32m by intercepting percolating surface water. Some water exists below the dee drainage system in compacted backfill, fractures in the marl, and along the concrete-bedrock contact. Removal of t water is accomplished by means of horizontal drains through the walls of structures and by pumping wells.

(c) Subsurface barrier.

A compacted silt blanket and slurry wal trench prevent infiltration of Ebro Riv water to the site.

Hydrologic studies were performed to provide a understanding of the ground water regime and i relation to foundation heave at Unit II, and t assess the effectiveness of existing water control measures. The studies included installation of piezometers and observation wells, performance of field and laboratory permeability tests, and water quality analyses

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The results of the studies indicate that ground water at the site occurs primarily in the compacted backfill with lesser quantities in terrace deposits, fractured marl, and in backfill concrete. The average water table is at about elevation 32m, or just below the deep drainage system. This ground water is isolated from the Ebro River by the subsurface barrier and by the essentially impervious marl bedrock (the average permeability of the upper 10m of marl is 1×10^{-4} cm/sec; below 10m the permeability averages 6×10^{-7} cm/sec). Ground water movement is toward the deep drainage system. Recharge to the ground water body is from construction water and infiltration of precipitation that bypasses the deep drains. Water balance calculations and tritium content analyses suggest that all of the water currently in the Unit II area has been recharged since construction began and has not infiltrated through the marl. Most waters at the site range in pH from 7.0 to 8.7 with a TDS of 3000 - 6000 mg/l and $CaSO_4$ (calcium sulfate) as the major dissolved constituent. Some NaCl water with pH in excess of 9.0 is found beneath the auxiliary building, and was probably formed by reaction with concrete.

CAUSE OF HEAVE

Heave data, mineralogical and swell tests, and ground water studies indicate that heave at Asco II is caused by expansive clay disseminated in the marl bedrock swelling due to the presence of water. The mechanism involved is the osmotic attraction of water by clay minerals with an expanding lattice structure (smectite). The principal clay mineral contributing to the swell is montmorillonite. The initial high rate of swell results from rapid surface adsorption of water. The low permeability of the rock allows only slow penetration of water which accounts for the initial surge of swell, followed by a uniform long-term rate.

The amount of heave exhibited at any particular locality on the site depends on the complex interaction of three primary factors:

(a) type and amount of smectite disseminated in the rock

(b) water availability, and porosity and degree of saturation of the rock

(c) applied loading

Using this qualitative model, the maximum amount of heave occurs where the rock contains significant amounts of smectite, where free water is available and can enter the rock through numerous fractures, and where the applied loading is light.

A reliable quantitative heave model cannot be postulated because of the complex interaction of such parameters as smectite chemistry and content, porosity, water availability, degree of rock saturation, and surcharge. However, a theoretical calculation of rock expansion under representative field conditions can be made to allow a rough comparison with the average amount of existing foundation heave. Oedometer test results indicate that an average load of 4 kg/cm^2 will inhibit swelling. Taking into account an average surcharge load of 2.7 kg/cm^2, it can be shown that only the upper 6m of rock is capable of heaving. Mineralogical analyses indicate that an average of 3 percent of the total rock is smectite, and that 75 percent of the smectite is oriented horizontally and thus capable of expanding vertically. It is believed that approximately one-half of the horizontally oriented smectite fills pore space and does not contribute to expansion. Further, interlayer water content studies showed that the average existing expansion of the smectite structure due to added water layers is approximately 200 percent. Using the above values and assuming that swell varies linearly with depth, the theoretical rock expansion (H) was calculated as follows:

$$H = 6\text{m} \times 3\% \times 75\% \times 50\% \times 1/2 \times 200\% = .068\text{m}$$

This average calculated foundation heave of about 70mm based on theoretical rock expansion compares reasonably well with measured heave at the site, and thus supports the conclusion that an average smectite content of as low as 3 percent can cause heave of the magnitude experienced.

ESTIMATES OF FUTURE HEAVE

In 1980, with approximately 4 years of heave records available and all structural loadings complete, studies were made to estimate future foundation heave at Unit II over the life of the plant. These estimates were necessary to assess the capacity of structures to resist heave effects and to evaluate measures taken to control heave. Upper and lower bound estimates were made. Case A, the upper bound estimate, assumed continued access of water to the marl during the life of the plant. This would be the case if no further water control measures were undertaken or if such measures were ineffective. Case B, the lower bound estimate, assumed complete water control by the end of 1980 which would effectively prevent further water infiltration at the site.

Forecasting ultimate heave based on a mathematical model which would accommodate the varying bedrock and ground water conditions, structure types, and foundation sizes and loads was considered impractical. The most useful approach was considered to be extrapolation based on measured data. Regression analysis was selected as the most appropriate method of estimating future heave based on past trends. The analysis was performed on 157 individual records for points located on the major structures. All data were computerized to facilitate regression analysis and plotting.

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Various types of curves were analyzed, including linear, exponential, power, and hyperbolic functions. The least squares method was used to find a best-fit representation for the measured data. It was concluded that the best representation of the past heaving patterns is generally a series of straight-line segments, the slope of which decreases with time. The number of segments in a series is, of course, somewhat arbitrary but the results indicate that this factor does not significantly affect the end results. It was found that a curve made of four straight-line segments would generally fit the observed data well for the four-year period from 1976 to 1979. An example of the curve-fit for the four years of data prior to 1980 for point EA-09 in the auxiliary building is shown in Figure 4. The slope of each line, B, represents the average heave velocity in millimeters per month over the 12-month period covered by the straight-line segment under consideration.

Estimates of future heave were based on extrapolation of the straight-line segments beyond the end of 1979. It was recognized that no single mathematical curve, even though it appears to fit four years of records, can be extrapolated with certainty forty years into the future. Accordingly, only upper and lower bound estimates of future heave were sought.

To obtain an estimate of the upper-bound additional heave under Case A, it was conservatively assumed that the pattern of straight-line segments observed in the past would continue in the future. Since the slope of these segments was decreasing, the average slope of the future records would range between a maximum equal to the slope of the last straight-line segment and zero. Consequently, the extrapolation to the year 2021 was based on a straight line having a slope equal to one-half of the slope calculated by regression analysis for the last record period.

To estimate the lower-bound additional heave under Case B, it was optimistically assumed that an impermeable layer, which would completely prevent any further water infiltration to the foundation rock, would be in place by the end of 1980. Owing to the delayed response of the expansive clays, it was further assumed that some movement would continue at a decreased rate through 1981 and would have effectively stopped by January 1, 1982. The value of additional heave for this case was thus obtained by extrapolation of two straight-line segments starting January 1, 1980 and January 1, 1981. The slope of each segment was estimated to have decreased by an amount equal to the reduction in slope observed between the last two regression lines. Figure 4 illustrates the extrapolation procedure for point EA-09 in the auxiliary building. The estimated additional heave values of 163mm and 13mm for Case A and Case B, respectively, were calculated as follows:

Case A: 41 years @ 1/2 x 0.663 mm/mo
$H = 41(0.663/2)(12) = \underline{163\ mm}$

Case B: Reduction in slope from last two regression lines = 0.082

$B = 0.663 - 0.082 = 0.581$ (for 1980)
$B = 0.581 - 0.082 = 0.499$ (for 1981)
$H = 0.581(12) + 0.499(12) = \underline{13mm}$

The results for Cases A and B are plotted in Figures 5 and 6 which show the estimated absolute heave between January, 1980 and January, 2021 under Case A, and between January, 1980 and January, 1982 under Case B. The maximum value calculated under Case A is 194mm, with high values generally occurring at the southwest corner of the site in the auxiliary and control buildings (Figure 5). Under Case B, the maximum value calculated is 19mm and the same locations are seen to be the most active (Figure 6). The maximum calculated differential heave under Case A, which occurs between the control building and control-turbine penetration is 31mm. The maximum differential heave at the same location is 6mm under Case B.

Subsequent to the regression analyses described above, additional data points through September, 1982 were analyzed and compared to the Case A

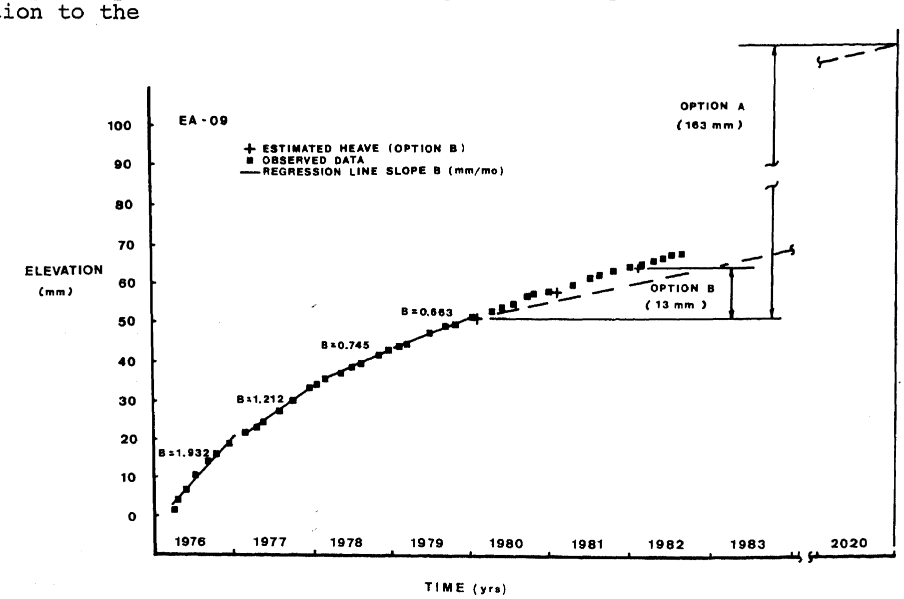

Fig. 4 Extrapolation Procedure for Point EA-09

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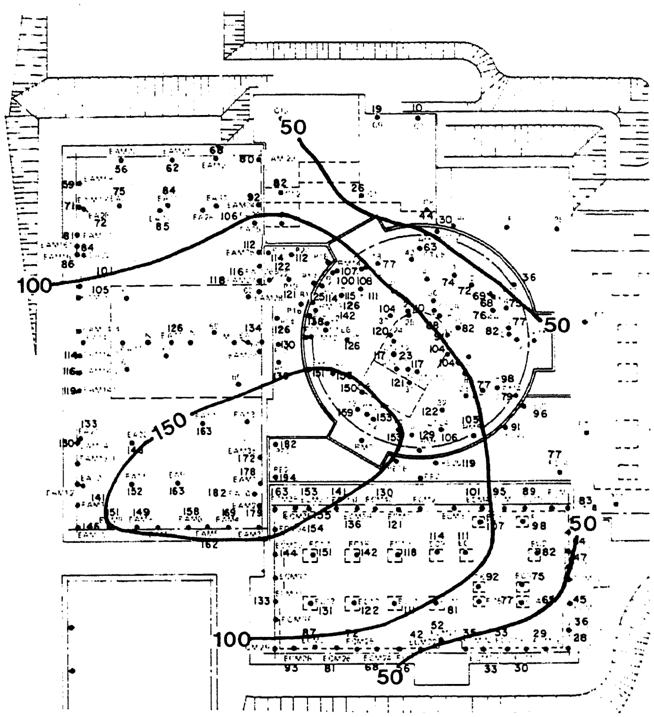

Fig. 5 Estimated Foundation Heave (mm) - Case A

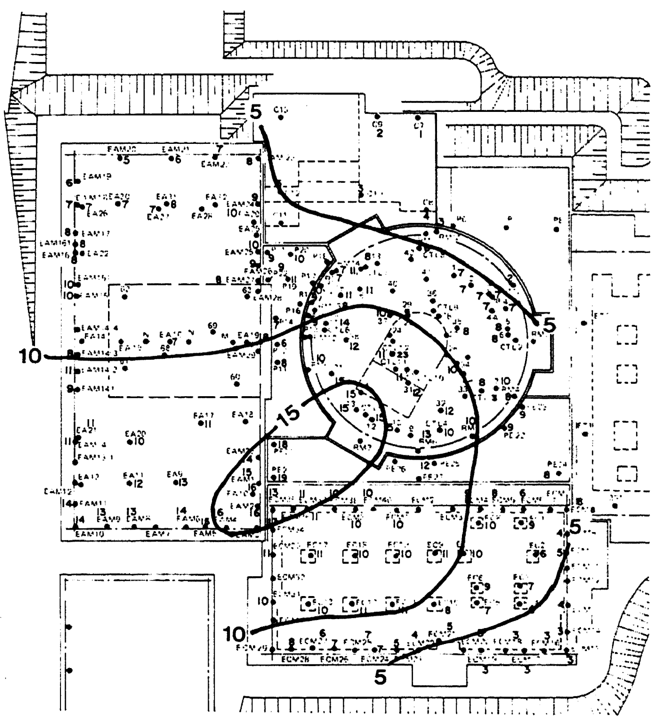

Fig. 6 Estimated Foundation Heave (mm) - Case B

and Case B estimates. A typical example of the 1980-1982 data is shown in Figure 4 for point EA-09. The comparison for all points indicated the following:

(a) There is general agreement between the heave calculated under Class B and the measured data, which indicates that foundation heave is generally decreasing at the site. Case B was in fact optimistic, but this is not surprising since full impermeabilization of the site was not achieved by the end of 1980.

(b) The estimated heave values under Case A appear to be quite conservative.

(c) The central part of the site shows a somewhat higher activity than previous records had indicated, while the south-western corner of the site, which was the area of most active swelling up to 1980, shows a significant decrease in activity. This may be due in part to the effectiveness of impermeabilization measures instituted in the southwestern part of the site.

(d) The maximum differential heave between buildings tends to be smaller than that estimated under Case B. This is due to the fact that swelling activity in the vicinity of the auxiliary building and control-turbine penetration has decreased while activity at the control building has increased somewhat.

EVALUATION OF STRUCTURES

Selected structures in the highest heave areas were analyzed to assess their capacity to resist heave generated loads and displacements. Heave values corresponding to Case A, the upper bound 40-year estimate, were used as applied deformations. Stresses resulting from a deformation analysis are generated as a result of the stiffness of the structure. As concrete cracks due to being forced into a prescribed deformation, the stiffness and corresponding stress in the vicinity of the crack is reduced. The displacement generated loads are thus self-limiting in nature and the resulting stresses are classified as secondary stresses. Accordingly, acceptance criteria used in the evaluation of shear walls was based on maintaining acceptable ductility levels, where ductility refers to the ratio of computed and yield strain of the reinforcement. For basemats, the ultimate moment capacities of each section were computed and compared with the heave induced moments.

Detailed studies were performed on the control and auxiliary building walls and basemats. A study of the heave displacement patterns indicated that other structures were either in low heave areas or would undergo rigid body displacement and thus would not experience structural effects. For shear walls, inelastic finite-element computer codes were used which allow consideration of the cracking of concrete and yielding of

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reinforcement. For basemats, the deformed shapes due to predicted heave displacements were computed and the resulting induced moments calculated. The calculated elastic moments were adjusted to account for the effects of concrete cracking.

The results of analyses of the most affected shear walls in the control and auxiliary buildings indicated that no more than about 50 percent of the concrete elements would crack and less than 1 percent of the reinforcement elements would yield. Maximum calculated member ductility ratios were all less than a range of 2 to 3 considered acceptable. Ductility ratios were conservatively calculated by taking the maximum total strain as the sum of the maximum strain from the heave analysis and the design load analysis without regard to location. Further, the design strains were conservatively assumed to be at 90 percent of yield strains.

Analysis of the most affected basemats indicated that the maximum percentage of the ultimate moment capacity of the sections utilized by heave induced moments ranged from 10 to 20 percent. This check demonstrated that there is sufficient reserve capacity to resist heave induced moments even under the conservative displacement conditions assumed.

CONCLUSIONS

1. The heave at Asco II is caused by swelling of expansive clays (smectite) in the presence of water. The principal clay mineral contributing to the swell is montmorillonite. This mineral is disseminated throughout the marl bedrock and constitutes on the average 3 percent of the volume of the rock. The resulting heave at any location depends on the type and amount of smectite in the rock, the availability of water, and applied loading. Wherever unsaturated smectite exists at a depth less than about 6m below the bedrock surface, the potential for additional swell exists.

2. Heave has generally decreased in all areas since completion of foundation loading. Upper-bound estimates of future heave based on past trends are considered to be quite conservative when compared to the presently observed trend of the heave data. Analysis of structures using deformation patterns based on the upper-bound heave estimates ensure that foundation heave effects are well accounted for in design.

3. The primary factor in controlling heave is the amount of water available to the foundation marls. Water control measures, such as surface impermeabilization and deep drains, have generally been effective in preventing access of water to the rock and thus attenuating heave rates.

ACKNOWLEDGEMENTS

This case history is based on the results of multi-discipline studies performed by individua too numerous to identify here. Their team effo during the investigation, testing, and analysis phases resulted in the successful resolution of the heave problem.

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