Investigation and Rehabilitation to Extend Service Life of DSS-13 Antenna Concrete Foundation

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It was noticed in 1977 that exposed surfaces of the reinforced concrete foundation of the DSS-13 26-meter antenna were exhibiting relatively light cracking. By 1980 the cracking had worsened to the point were it was decided that an investigation should be undertaken to establish the cause and, as needed, devise a repair technique that would maintain the serviceability of the antenna. Core samples were obtained from the concrete and various laboratory tests conducted. In-place nondestructive type tests were also performed. The tests established that the concrete was deteriorating because of alkali-aggregate reactivity. This is a phenomenon wherein certain siliceous constituents present in some aggregates react with alkalies in the portland cement to produce a silica gel which, in turn, imbibes water, swells, and cracks the concrete. After a thorough structural analysis, a rehabilitation scheme was devised and installed. The scheme consisted of a supplemental steel frame and friction pile anchored grade beam encircling the existing foundation. This system provides adequate bracing against base shear and overturning due to seismic loading. Larger cracks were sealed using a pressure injected two-component epoxy.

I. Introduction

An extensive investigation was undertaken by the Ground Antennas and Facilities Engineering Section to determine the cause of severe cracking observed in the DSS-13 (Venus-Station) antenna reinforced concrete foundation and to design structural steel bracing to extend the useful life of the antenna. The antenna is one of several based at the Deep Space Communications Complex, Goldstone, California, which is owned by NASA and operated by the California Institute of Technology's Jet Propulsion Laboratory.

The investigation involved examination of the original concrete design, in-place testing and testing of cores taken from the interior and top surfaces of the foundation. The tests included ultrasonic pulse readings, petrographic examination, standard compression tests, standard tensile splitting tests, surface condition survey, determination of rebound number, pull-out tests of anchor bolts, and monitoring of crack widths under dynamic and wind loads. The rehabilitation found necessary to extend the useful life of the antenna foundation was established and has been successfully completed.

II. Structure Description

The DSS 13 antenna is a 26-meter-diameter parabolic reflector with an azimuth-elevation mount. The base frame consists of eight steel columns that taper outward as they descend to the concrete foundation where they are at a radius of 3.98 m (13 ft, ½ in.) at the top of the bearing plates. Each column is anchored into the foundation with four 57.15-mm-(2¼-in.-) diameter anchor bolts that extend 2.13 m (7 ft, 0 in.) into the concrete.

The foundation is 10.67 m (35 ft, 0 in.) square in plan. Figure 1 shows a partial plan and section. The top of concrete is slightly above adjacent grade and extends to a depth of 3.05 m (10 ft, 0 in.) as measured around its perimeter. In the center is a 3.66-m- (12-ft-0-in.-) diameter pit, 3.96 m (13 ft, 0 in.) deep. The footing under this pit is 0.76 m (2 ft, 6 in.) thick. The main reinforcing steel in the 3.05-m- (10-ft-0-in.-) thick section consists of No. 8 (25.4-mm- (1-in.-) diameter) bars at 152.4 mm (6 in.) on center each way, top and bottom. The reinforcing steel in the 0.76-m- (2-ft-6-in.-) thick section under the pit is also No. 8 bars at 152.4 mm (6 in.) on center each way on the bottom including the sloping transition, but the top bars in this area are No. 4 bars (12.7-mm- (½-in.-) diameter) at 0.305 m (12 in.) on center.

The only vertical reinforcing steel in the main 3.05-m- (10-ft-) thick section of the footing consists of No. 4 bars at 0.305 m (12 in.) on center each way in the outer face and in the face of the 3.66-m- (12-ft-) diameter pit.

III. Foundation Concrete Materials

The concrete foundation was placed during May and June of 1962. The mix design (Table 1) called for a 25.0-mm (1-in.) maximum aggregate, 334.6 kg/m^3 (6.0 sacks per yd^3) of cement and a maximum allowable water content of 5.33×10^{-4} m^3/kg (6.0 gal per sack) of cement. The aggregate source (Table 1) was Barstow, California; no records are available indicating the cement brand or type used. No admixtures were used.

The mix design was for a 24.13-MPa (3500-psi) compressive strength although the structural design was based on 20.68 MPa (3000 psi). Only five field test cylinders were taken during construction. This was inadequate. The volume of foundation concrete is on the order of 436 m^3 (570 yd^3) and the American Concrete Institute Building Code requires that a minimum of 16 field cylinders should be tested for this volume.

Three of the 152-mm- (6-in.-) diameter by 304.8-mm (12-in.) high field test cylinders had an average compressive strength of 17.31 MPa (2490 psi) at age 7 days. Two cylinders at age 28 days had an average compressive strength of 24.75 MPa (3590 psi).

IV. Structural Loading

The horizontal load, due to wind, on the main reflector is 311,375 N (70 kips) and on the steel base, 44,482 N (10 kips). The dead load of the main reflector is 1,334,467 N (300 kips) and of the steel base, 756,198 N (170 kips). A design check by a licensed structural engineer indicated there were no deficiencies in the original as-built structure.

V. Condition of the Structure

Cracks of various types have developed on almost all exposed surfaces of the foundation. "Map" or "pattern" cracking has developed over all exposed surfaces. The "map" cracks are most prominent on the upper horizontal surface of the foundation, with little or no difference between the exterior area and the area enclosed within the base. The "map" cracking, although moderate to very slight, appears on all surfaces within the pit area. The major system of cracks is a series of generally horizontal cracks, located on the vertical surfaces in the pit area. These cracks have a vertical spacing ranging from about 0.305 m (12 in.) to 0.457 m (18 in.). A majority of these cracks have a width on the order of 0.25 to 1.27 mm (0.010 to 0.050 in.), but two cracks have widened to approximately 12.7 mm (0.5 in.). Figure 2 is a photo taken of one of these cracks. An inspection pit was dug along the westerly side of the foundation exterior extending to its full depth of 3.05 m (10 ft). The crack pattern there was generally similar to that exposed in the interior pit described above.

VI. Investigation Program

Investigation of the foundation began in January 1980, when it was becoming apparent that areas of structural distress were developing. Cracking was first reported in 1977 by station personnel, but no untoward problem was thought to exist at that time because the cracking was relatively light and superficial in nature, and cracks often form in normal concrete due to thermal changes or drying shrinkage. The cracks, however, progressed to a point where, in 1980, an investigative program was initiated. The investigation included visual inspection, measurement of crack widths at selected locations, pull-out tests on two anchor bolts, monitoring of crack width during periods of high winds, and testing of concrete cores.

The most important information in concrete investigative work is obtained from the testing of cores. Tests on core samples provide a direct determination of absolute strength and elastic properties. Specimens obtained from cores permit petrographic study that can possibly establish the cause of cracking. Horizontal 152-mm- (6-in.-) diameter cores were taken in August, 1980, from the vertical wall in the pit area, which extended 0.61 to 0.91 m (2 to 3 ft) into the footing. Vertical 50.8-mm- (2-in.-) diameter cores were also obtained from the outdoor top of the foundation at the locations shown in Fig. 1. They extended to a depth on the order of

2.90 m (9 ft, 6 in.). The core locations were generally established by visual inspection to provide information on areas showing greatest distress as well as those of less distress.

VII. Description of Concrete Test Cores

The cores contained fine cracks of random orientation and slightly larger cracks of a generally horizontal orientation. Photographs of the two vertical cores in core storage boxes are shown in Fig. 3 and 4. Reinforcing steel was cut in three cores. No corrosion was observed on the steel and the bond between the steel and the concrete was tight. No rust stains have been observed on concrete surfaces near the cracks. This indicates that no corrosion of the steel is occurring.

VIII. Description of Concrete Tests

To determine the general condition of the concrete, the following tests were made:

(1) Ultrasonic pulse velocity tests to determine depth of major surface cracks.

(2) Standard compression tests.

(3) Petrographic (microscopic analysis) examination of cores to establish aggregate mineral composition.

(4) Standard tensile splitting test.

(5) Determination of rebound number.

Pulse velocity tests were performed in accordance with Ref. 1. The test, in this project, was used to measure the depth of the crack having the largest surface width. The test was done in accordance with procedures set forth in Ref. 2.

The basic principle of crack detection by pulse velocity tests is: If a crack is of appreciable width and is of considerable depth perpendicular to the test path, the path of the pulse will be blocked and no signal will be received at the receiving transducer. If the depth of the crack is small compared to the distance between the transducers, that is, the path length, the pulse will pass around the end of the crack and a signal will be received at the transducer. However, in doing so it will have traveled a distance longer than the straight line path upon which the pulse velocity computations are based. The resulting calculated pulse velocity will then be low in comparison with that through uncracked concrete in the same vicinity. The difference in the pulse velocity is then used to estimate the path length and hence the crack depth. Thus if the transducers are equidistant (x) from each edge of the crack, as shown in Fig. 5, the depth h of the crack can be obtained as follows:

$$\text{longitudinal pulse velocity} = \alpha$$

$$\text{distance traveled in concrete without crack} = 2x$$

$$\text{distance traveled in concrete with crack} = 2\sqrt{x^2 + h^2}$$

$$T_c^2 \text{ (in concrete with crack)} = \frac{4h^2 + 4x^2}{\alpha^2}$$

$$T_s^2 \text{ (in concrete without crack)} = \frac{4x^2}{\alpha^2}$$

$$h = x\sqrt{\frac{T_c^2}{T_s^2} - 1}$$

where

T_c = travel time around the crack

T_s = travel time along the surface of the same type of concrete without cracks

Compression tests were performed in accordance with Ref. 3. All cores were tested dry although ASTM C 42 provides that cores be tested promptly after being stored in lime-saturated water for 40 hours. Reference 4 states that cores shall be tested dry if service conditions in the structure are dry.

A petrographic study of random segments of the test cores was made microscopially to establish the types of minerals present in the aggregates and to determine if alkali-aggregate reactivity was occurring.

Splitting tensile strength was determined by testing core samples in accordance with Ref. 5.

Rebound number of the in-place concrete was measured on the interior face of the foundation in the pit area in accordance with Ref. 6. Three representative areas were tested.

IX. Test Results

A. Sonic Tests

The sonic test was used to measure the depth of the largest surface crack exposed in the interior pit face. The depth was determined to be on the order of 0.69 m (27 in.). Values obtained using this method were supplemented with other observations made on actual cores on cracks because it has not yet been established how wide a crack must be to significantly increase the transmission time.

B. Standard Compression Tests

Six 101.6-mm- (4-in.-) diameter cores were obtained in the pit area walls and delivered to an independent materials testing laboratory for testing. Due to the friability of the samples it was possible to cut and trim only one of the cores for compression testing. The ultimate compressive strength of the one core tested was 22.03 MPa (3195 psi). This one test result is obviously very selective and can be considered to represent an approximate upper limit for compressive strength in that it was the only one segment out of six cores sound enough to enable fabrication of a test sample. Indeed, portions of some cores could be crumbled by finger pressure. The one compressive strength test value obtained is below the design mix concrete strength of 24.13 MPa (3500 psi), but slightly greater than the structural design strength of 20.68 MPa (3000 psi). It must be assumed that the strength of the concrete in much of the mass might be well below the design strength.

C. Petrographic Examination

Core segments were petrographically examined by Dr. Richard Merriam, consulting engineering geologist, in September 1980. His examination revealed the aggregate consists largely of volcanic rocks, many of which are of approximately andesitic composition and have partly glassy groundmasses. Such rocks are known to be reactive with cement alkalies. Dark "reaction rims" were observed around the periphery of some of the broken andesitic aggregate particles as well as deposits of white silica gel in air voids and within cracks. Both are features of concrete experiencing alkali-aggregate reactivity, and Dr. Merriam concluded that the deterioration was due to alkali-aggregate reaction. About one year later, in October 1981, Mr. David Stark, Principal Research Petrographer of The Portland Cement Association, Skokie, Illinois, visited the Venus antenna to investigate our alkali-aggregate reactivity because none had previously been reported in the Barstow area. His on-site inspection supplemented by his petrographic study of JPL-supplied core segments led him to agree with Dr. Merriam that the Venus antenna foundation concrete was indeed experiencing alkali-aggregate reactivity.

D. Standard Tensile Splitting Test

The splitting tensile test was performed on two test cores by Twining Laboratories of Long Beach, California. The values obtained were 2.55 and 2.96 MPa (370 and 430 psi). The tests were selected to determine the influence of the reactivity microcracking on tensile strength. The values obtained were considered somewhat low by Twining Laboratories. Further, the percentage of broken aggregate exposed on the split sample faces was very small, which indicates that the bond of the aggregate to the paste matrix was low.

E. Rebound Number

The compressive strengths obtained using the rebound method were above 35.85 MPa (5200 psi). The rebound hammer, in this case the Schmidt type, is principally a surface hardness tester and there is little apparent theoretical relationship between the strength of concrete and the rebound number obtained using the hammer. Within limits, however, empirical correlations have been established between strength and the rebound number but there is a wide degree of disagreement among various researchers concerning the accuracy of the estimation of strength from rebound readings. The consensus among users is that it is useful only as a rough indication of concrete strength. In the case of the Venus foundation concrete, the hammer yielded much higher values of compressive strength than those obtained from tests of core samples taken immediately adjacent to hammer test areas. It is therefore believed that no conclusions can be drawn from the hammer data.

X. General Condition of the Concrete

The test results indicated the following condition of the concrete:

(1) The concrete is of questionable to poor quality.

(2) Compressive strength is below the design compressive strength.

(3) The aggregate is very reactive.

XI. Cause of Cracking

When all the test results were reviewed, the cracking was established to be principally the result of the alkali-aggregate reactivity. No corrosion of reinforcing steel was observed in the cores and no rust stains were observed on the concrete near cracks. Slight corrosion may be present but not in sufficient amounts to be disruptive. Corrosion is a major concern because the corrosion products occupy 2.2 times as much volume as the original metal and may develop pressures up to 32.41 MPa (4700 psi). This is considerably greater than the tensile strength of concrete which is generally less than 3.45 MPa (500 psi), and disruption of the concrete ensues. Once the corrosion begins, it continues as long as oxygen and moisture can reach the reinforcing steel. The dry desert atmosphere has no doubt helped in keeping corrosion to less than that needed to noticeably crack the concrete.

XII. Alkali-Aggregate Reactivity

The phenomenon of alkali-aggregate reactivity involves a chemical interaction produced by certain siliceous constituents present in the aggregates with alkalies in the cement. The reaction produces silica gel, which then imbibes water from the surrounding concrete and swells. The resulting volumetric expansion damages the concrete by causing intense internal microfracturing with an attendant reduction in the strength of the concrete and a diminution of its elastic properties.

T.E. Stanton of the California Division of Highways first recognized the serious cracking and deterioration of a concrete pavement in the Salinas Valley, California, in 1938, and in 1940 he published a paper (Ref. 7) on the influence of cement alkalies on certain aggregates. Based on his work, the Division of Highways changed its cement specifications to include a top limit of 0.50 percent total alkali content (percent Na_2O + 0.658x percent K_2O) for regions where reactive aggregates were used in making concrete. Subsequently, an alkali content of 0.6 percent of equivalent Na_2O was accepted in the United States as an upper limit for cement when used with reactive aggregates. No records exist to indicate that the cement used for construction of the Venus antenna foundation was certified as being a "low-alkali" type. Manufacturers in the Mojave Desert area have reported verbally that all of their cement production, regardless of type, has met the 0.6-percent limit since the early 1950s, which would cover the period of construction of DSS 13. Thus the best preventative practice of the time was followed, consciously or not, during construction. There are indeed no indications that the Barstow area concrete industry was aware, at the time of construction, that local aggregates were reactive.

In recent years, some investigators have come to realize that the use of low-alkali cement is not always effective in controlling deleterious expansion and in many cases only slows the reaction.

David Stark of the Portland Cement Association first reported in 1979 (Ref. 8) that field and laboratory observations indicated that certain glassy volcanic aggregates used with low-alkali cements can react deleteriously. Stark identifies the reactive volcanic materials as being of andesitic to rhyolitic composition. These material types were found in significant amounts in core samples taken from the Venus foundation.

XIII. Rehabilitation Program

The results of the investigation indicated that the useful life of the Venus antenna foundation could be successfully extended if a rehabilitation program were developed and implemented. The design approach assumed that the foundation concrete was satisfactory to carry the pedestal's vertical dead and live loads even though cracked, but not its base shear and/or uplift loads.

Structural analysis determined that the antenna pedestal base does not develop uplift from wind loading, but from seismic loading described in Ref. 9. To restore the effectiveness of the existing foundation, it was considered necessary to provide only enough additional structure to resist uplift and base shear loads. The foundation is still capable of performing its function of spreading the concentrated antenna base dead and live loads at its top surface over the larger area of the foundation bottom and thereby distribute the load to a bearing pressure allowable for the bearing or founding soils. The required resistance to uplift was provided by installing a structural system composed of steel-framed braces attached to each of the eight sloped steel columns in the antenna base core. The outer lower end of each brace is anchored to a large reinforced concrete grade beam, which, in turn, is anchored by a system of cast-in-place concrete friction piles. A plan and partial section of the structural system is shown in Fig. 6. Construction started July 5, 1983, and was completed, August 4, 1983. Figure 7 shows the installed system.

In addition to the installation of the braces, all accessible cracks, greater than approximately 1.59 mm ($^1/_{16}$ in.) wide, were pressure injected with a two-component epoxy adhesive to seal the surface and stop moisture intrusion into the reinforcing steel.

All aggregate used in the new concrete work was obtained from the Owl Rock Co. pit located near San Bernardino, California. Petrographic study of this material showed it to be completely free of minerals known to be reactive.

XIV. Monitoring Program

Although the useful life of the Venus Antenna has been extended, periodic inspections will be required to observe the formation of new cracks or widening of the injected cracks. Further cracking is to be expected in that the alkali-aggregate reaction can continue for long periods of time before stopping. As new cracks occur, or old ones widen, additional epoxy adhesives will be injected. Thus the useful life of the antenna can be prolonged.

References

1. *Standard Method of Test for Pulse Velocity Through Concrete.* ASTM C 597, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1982.
2. Malhotra, V. M., *Testing Hardened Concrete: Nondestructive Methods.* Monograph No. 9, American Concrete Institute, Detroit, Michigan, 1976.
3. *Standard Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete.* ASTM C 42, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1982.
4. ACI Committee 318, *Building Code Requirements for Reinforced Concrete.* ACI Standard 318, 8th Printing, American Concrete Institute, Detroit, Michigan, 1976.
5. *Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens.* ASTM C 496, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1982.
6. *Test for Rebound Number of Hardened Concrete.* ASTM C 805, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1982.
7. Stanton, T. E., "Expansion of Concrete Through Reaction Between Cement and Aggregate," *Proceedings of the American Society of Civil Engineers*, Vol. 66, 1940, pp. 1781-1811.
8. Stark, D., "Alkali-Silica Reactivity: Some Reconsiderations," *Cement, Concrete and Aggregates*, CCAGDP, Vol. 2, No. 2, Winter 1980, pp. 92-94.
9. *Uniform Building Code, Seismic Zone IV.* International Conference of Building Officials, Whittier, California (latest issue).

Table 1._Supporting contractors

Name	Service
Pacific Materials Laboratory, Inc., Bloomington, Calif.	Concrete mix design
Concrete Materials Co., Barstow, Calif.	Source of aggregate

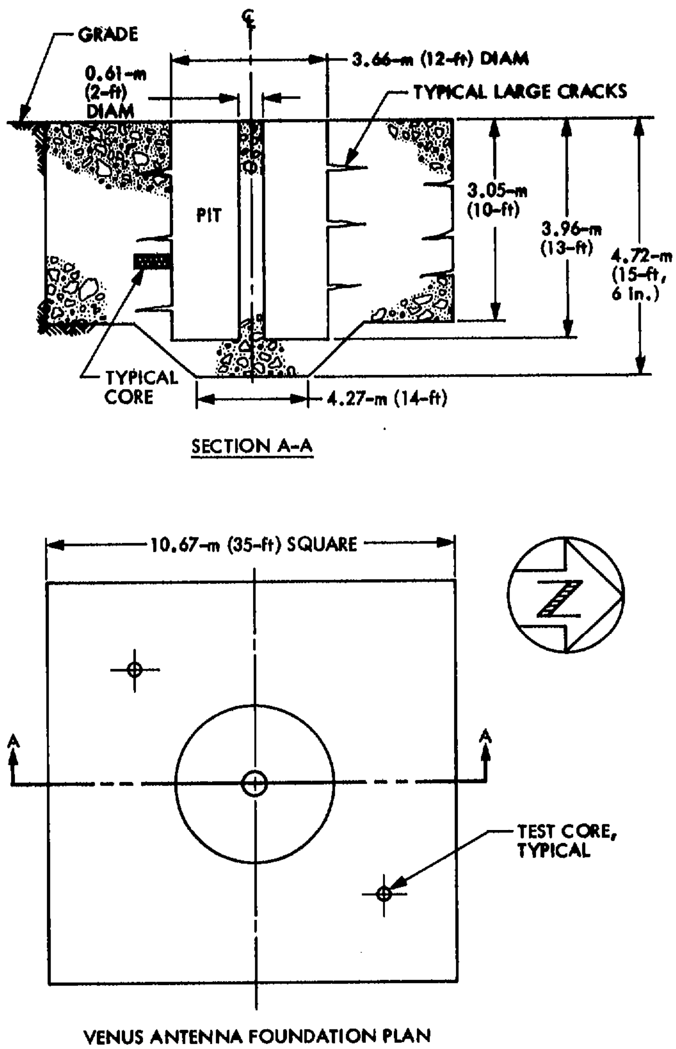

Fig. 1. Partial plan and section of original foundation

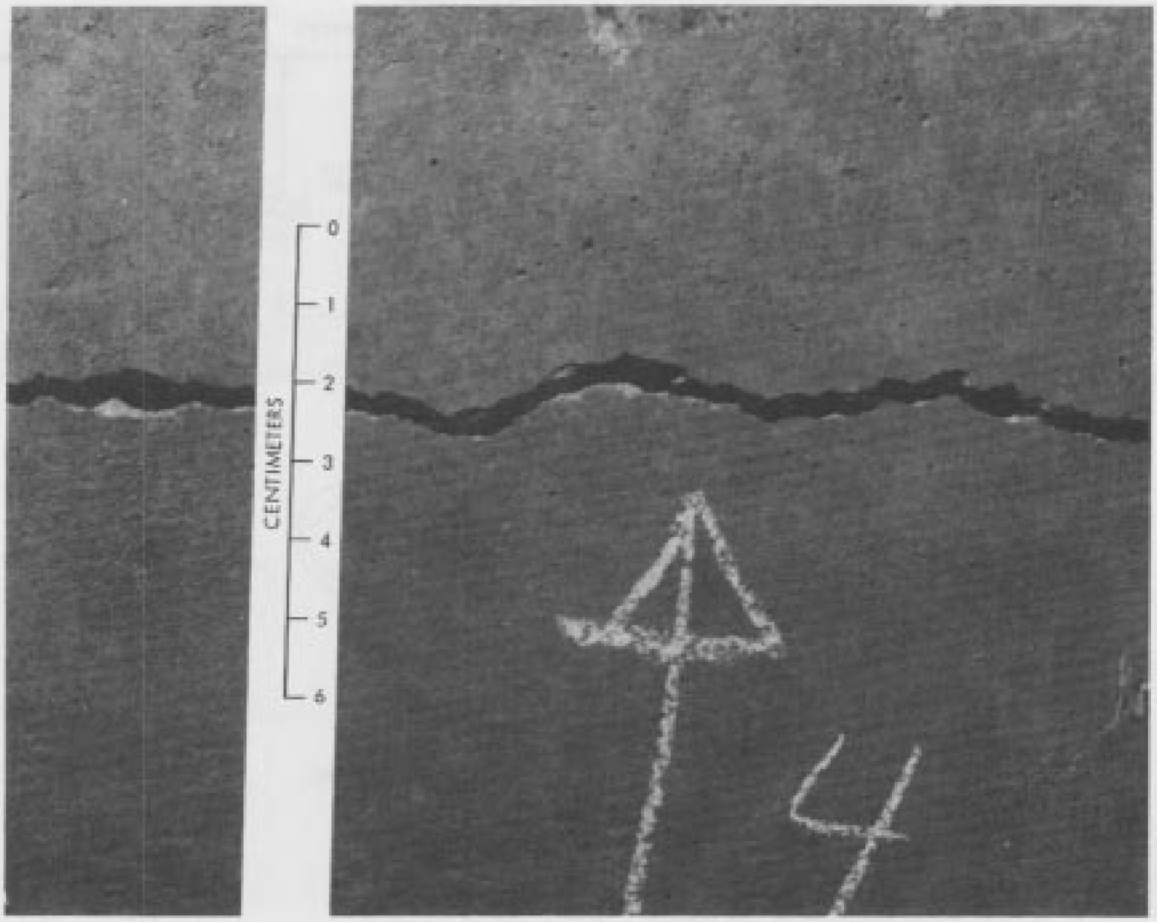

Fig. 2. Crack in pit wall

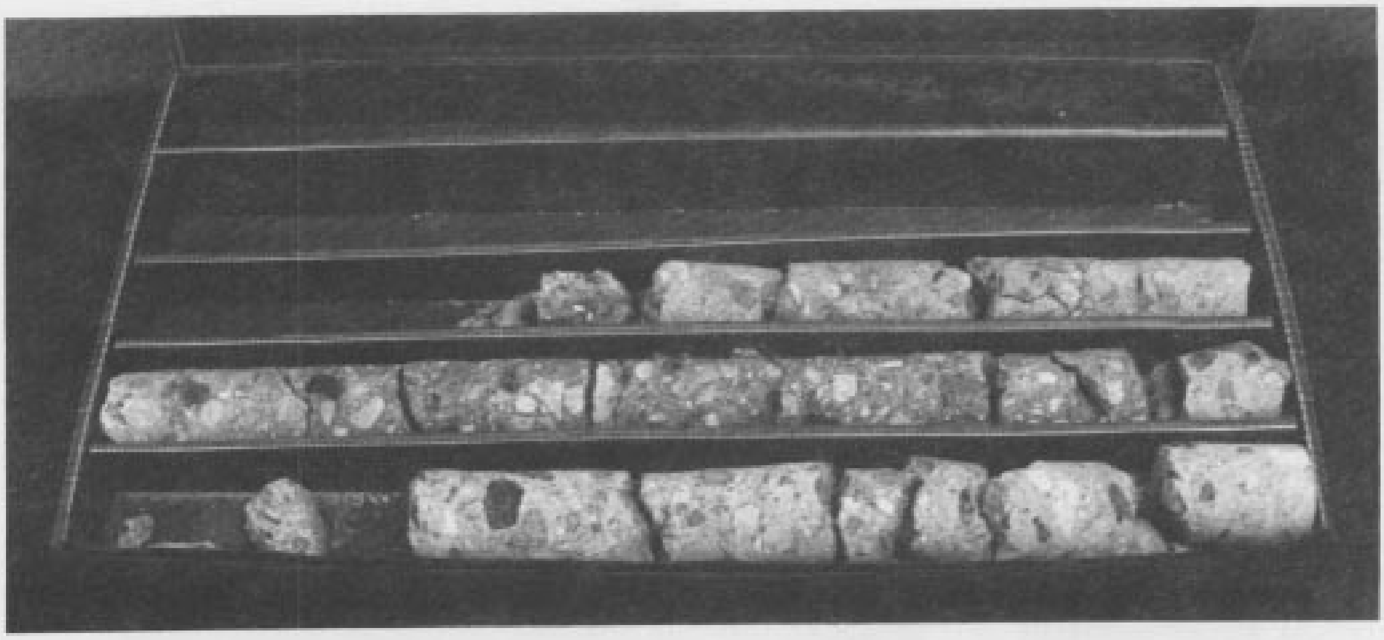

Fig. 3. Vertical core No. 1

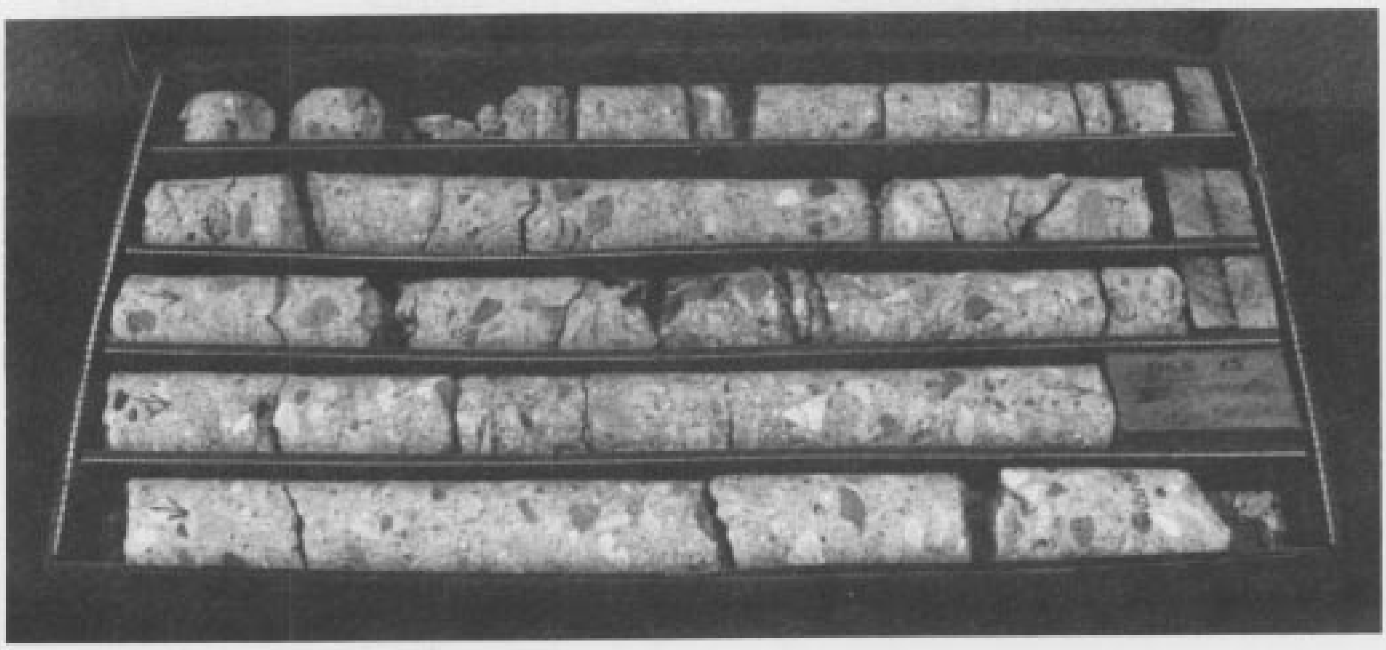

Fig. 4. Vertical core No. 2

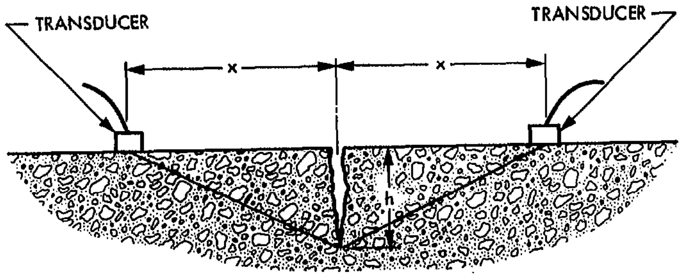

Fig. 5. Measurement of crack depth

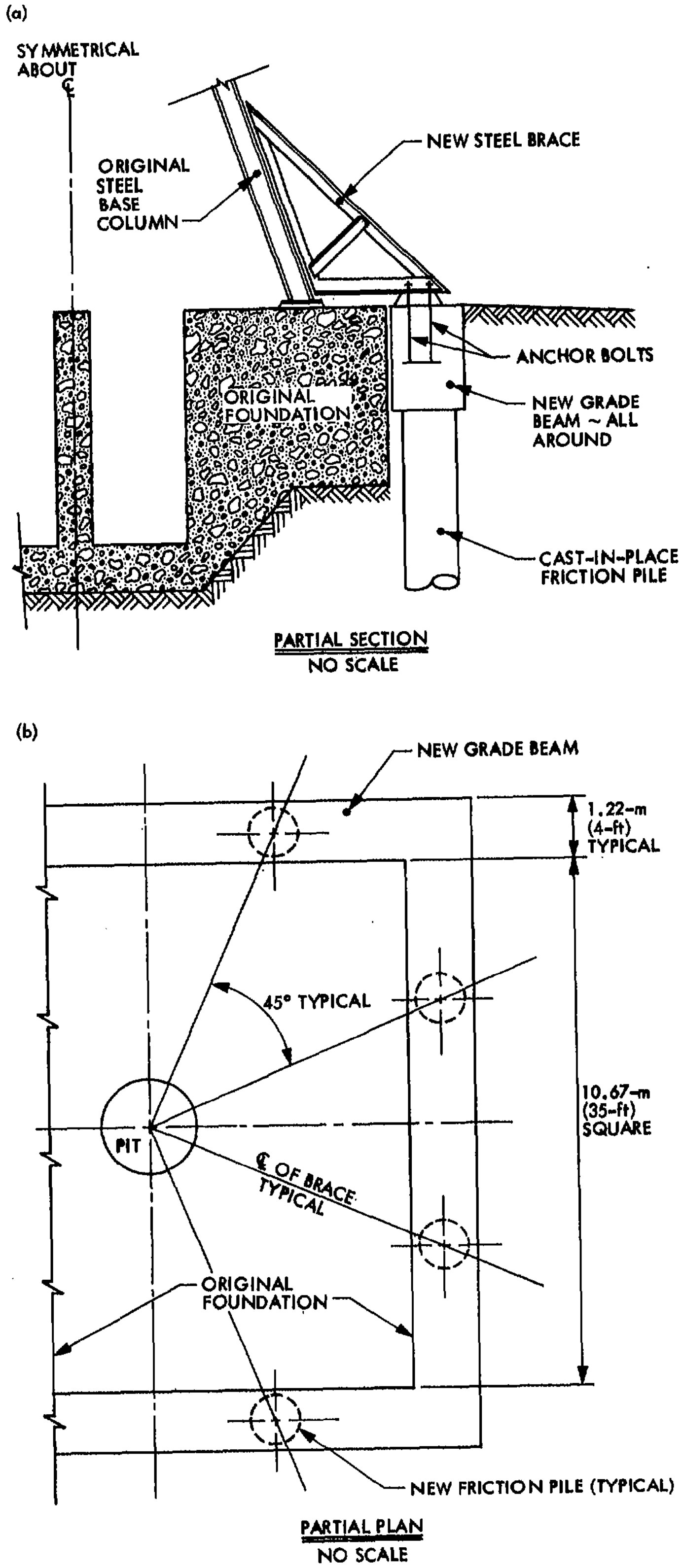

Fig. 6. Repair scheme (no scale): (a) partial section; (b) partial plan

Fig. 7. The installed system