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Hi-Tech Testing, Evaluation & Repair of Earthquake-Damaged Concrete Structures

Authors: Allen G. Davis*, W. Gene Corley* and Claus Germann Petersen**

ABSTRACT

After severe earthquake damage to the infrastructure in any community, managers responsible for damage assessment and reconstruction policy prioritization are faced with an extremely difficult, if not overwhelming task. Partially damaged buildings and structures have to be evaluated for their serviceability and their restoration potential. Rapid assessment techniques and programs have to be put in hand. However, "rapid" does not imply slipshod inspection, and as full an evaluation as possible of the structure's state is required for full confidence in subsequently selected repair schemes. Also, decisions have to be made about possible upgrading of older structures to meet seismic codes in place at the time of evaluation.

In recent years, the development of reliable nondestructive and partially invasive testing techniques has presented the opportunity to collect large amounts of data on a given structure, with minimal time and disruption to the users of the facility being tested. The authors of this paper have participated in this development, and a case history of the application of new, hi-tech testing methods to problems encountered in structures damaged by seismic activity are presented here, together with brief descriptions of some of these new testing methods. A review is also made of the problems encountered by parking structures during the Northridge, California Earthquake of 1994.

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1. INTRODUCTION

After severe earthquake damage to the infrastructure in any community, managers responsible for damage assessment and reconstruction policy prioritization are faced with an extremely difficult, if not overwhelming task. Partially damaged buildings and structures have to be evaluated for their serviceability and their restoration potential. Rapid assessment techniques and programs have to be put in hand. However, "rapid" does not imply slipshod inspection, and as full an evaluation as possible of the structure's state is required for full confidence in subsequently selected repair schemes. Also, decisions have to be made about possible upgrading of older structures to meet seismic codes in place at the time of evaluation.

In recent years, the development of reliable nondestructive and partially invasive testing techniques has presented the opportunity to collect large amounts of data on a given structure, within a minimal time frame and minimal disruption to the users of the facility being tested. The authors of this paper have participated in this development, and can testify to the success of projects where some of these new, hi-tech testing methods have been applied to problems encountered in structures damaged by seismic activity.

One of the hindrances to residual life assessment of a damaged concrete structure is that there is often no full record of its as-built condition and performance. Construction inspectors' records usually provide information on volumes of concrete poured and cylinder compressive strengths, but tell little about the factors that control concrete durability such as the concrete in-place density and the depth of steel reinforcement cover. Elements such as buried foundations are often not well documented during construction. Engineers evaluating them at a later date in the life of the structure have to spend considerable time and money, while often leaving a large degree of uncertainty about the structure's condition at the end.

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Judicious use of nondestructive testing and evaluation (NDT&E) in conjunction with more routine procedures can reduce the effort and cost in damaged structure evaluation, as well as giving a higher level of confidence in that evaluation. In the case of older structures with no base line survey, NDT can present the opportunity to widen the database while keeping the study budget down to reasonable levels.

The American Concrete Institute (ACI) Committee on Nondestructive Testing (Committee 228) has produced recommendations for the use of NDT in evaluation surveys for both compressive strength measurement and concrete properties related to durability [1,2].

Many NDT and partially invasive methods exist for testing concrete. As well as the two ACI references quoted above, Malhotra and Carino [3] describe the principles, limitations, and applications of most of the methods available today. The introduction of the portable computer and real-time data acquisition using small analog-to-digital cards in the portable PC has boosted data quality, as well as reducing the personal factor in data interpretation. Large quantities of test data can now be acquired and stored for future analysis in the course of a single day on site.

Working against this progress, test equipment has become more complex and expensive, with the result that not all testing companies own the full range of testing methods. It is a natural tendency for a testing company to suggest that all problems can be solved using their available test methods. Training in the use of test systems is imperative. Any NDT program must give the engineer a level of confidence at least as great as from coring and visual inspection alone. The investigator should also supplement knowledge and understanding of the structure by testing a larger volume.

2. NDT FOR CONCRETE STRENGTH DETERMINATION

By definition, NDT can not determine the strength of concrete directly, since all truly nondestructive tests measure material properties other than the ultimate strength. However, correlations between strength and NDT are possible, and the high cost of determining concrete strengths in existing structures by coring alone makes a combination of NDT and limited coring very cost effective. Whenever possible, the NDT survey should be carried out immediately before any coring, in order to establish the optimal number and location of core samples for correlation. Table 1 presents a summary of the more commonly practiced tests for strength evaluation.

The most widely used of these tests are the rebound hammer and pulse velocity (UPV). For both tests, no single correlation curve exists between the NDT parameters measured (rebound number or pulse velocity) and in-place concrete strength. Factors such as aggregate type, size and grading, cement type, moisture condition of the concrete and carbonation of the surface considerably affect any NDT parameter relationship. However, when the concrete mix specifications are known, correlations can be established with a reasonable degree of confidence. The probable degree of accuracy using the rebound hammer in this way is +/- 25% [1], with slightly improved accuracy with pulse velocity.

The semi-destructive CAPO-Test (Cut and Pull-Out Test) measures the compression strength directly at a depth of 25 mm. Using a general correlation tested over the past 20 years for a wide range of concrete mixes [4, 5], the accuracy for estimating in-situ strengths is within +/- 10%. This can be reduced to within +/- 5% by using a specially prepared correlation method.

TABLE 1 - NDT Methods for Strength Determination

TEST	MEASURED CONCRETE PROPERTIES	STANDARD
1. Rebound Hammer	Hardness	ASTM C 805
2. Windsor Probe	Penetration Resistance	ASTM C 803
3. Pulse Velocity	Modulus & Density	ASTM C 597
4. Break-off	Flexural Strength	ASTM C 1150, BS 1881, Pt201 (UK)
5. Pull-out (Capo-Test)	Compressive Strength	ASTM C 900; DS423.31(Denmark); BS1881: Part 207 (UK); CEN/TC104/SCI/TG8:187
6. Combined	At least two methods - usually #1 and #3	-

3. NDT FOR CONCRETE DURABILITY AND INTEGRITY

The most rapid development in NDT&E in recent years has been the improvement in the measurement of the durability and integrity of concrete structures. Many physical and electrochemical methods have been adapted for this purpose. Table 2 summarizes the NDT methods currently available, and references [2] and [3] describe these procedures in detail. The relatively recent increase in the application of NDT procedures has occurred in spite of a lack of testing standards for the majority of methods in Table 2.

Traditionally, quality assurance of concrete placement and condition evaluation of structures has been performed by visual inspection and intrusive sampling for strength tests. This does not provide direct data on in-place concrete durability and integrity. Methods outlined in Table 2 can give information on:

- member dimensions (example in Figure 1(a)),
- cracking, delamination and debonding (examples in Figures 1(b) and 1(c)),
- depth of surface opening cracks (example in Figure 2),
- consolidation, voids and honeycombing,
- steel reinforcement location (cover) and size,
- seismic, freeze-thaw, fire, chemical and other damage.

TABLE 2 - NDT Methods for Durability and Integrity

METHODS & PRINCIPLES	FINDINGS & APPLICATIONS
Ultrasonic Pulse Velocity (UPV) - Travel time of an ultrasonic pulse over a known path length.	Gives relative quality of the concrete, the faster the pulse, the better.
Impact-Echo - Stress waves echoed from opposite side of member or from defect are monitored. Frequently analysis gives distance to reflector.	Locates defects such as delaminations, voiding, honeycombing and cracking. Maximum element thickness is about 2 m.
Spectral Analysis of Surface Waves (SASW) - Impact generates surface waves, which are monitored by two receivers. Elastic constants of layers are determined.	Determines the stiffness profile of a pavement system or the depth of deteriorated concrete.
Sonic-Echo - Impact on surface, with receiver monitoring reflected stress wave and travel time.	Determines length of and defects in deep foundations (piles and drilled shafts).
Impulse Response (Mobility) - Similar to Sonic-Echo, with extra signal processing of received signal and impact force in the frequency domain.	Determines length and depth to defects in deep foundations. Measures stiffness and mobility of structural elements such as pavements and walls.

Cross-hole Sonic Logging - Uses UPV with transducers in tubes cast in deep foundations or holes drilled after construction.	Determines concrete quality along the length of the profile measured. Used in drilled shafts and slurry trench walls.
Parallel Seismic - Receiver placed in borehole next to deep foundation. Foundation struck by hammer and signal from receiver at different depths recorded.	Determines the depth of the foundation and its quality.
Gamma-Gamma Logging - Gamma radiation used to measure concrete density in either cross-hole or backscatter mode.	Verifies concrete density for placement in drilled shafts and slurry wall foundations.
Infrared Thermography - Heat conduction properties of the concrete are measured by differences in surface temperatures in correct ambient conditions.	Locates delaminations in pavements and bridge decks, as well as moist insulation in buildings.
Radar - As Sonic-Echo, but electromagnetic waves are used. Interfaces between materials with different dielectric properties are detected.	Locates metal embedments (reinforcing) and voids beneath pavements, indicates thickness of elements.
Covermeter - Measures location and depth of steel inclusions using magnetic induction.	Locates reinforcing steel, as well as its depth and size.
Electric Half-Cell - Measures the negative potential between steel reinforcement and the concrete surface. Larger negative potential means increasing corrosion.	Allows the mapping of steel reinforcement corrosion in reinforced concrete elements.

The following case histories describe the application of some less well-known methods for condition surveys.

4. CASE HISTORIES

4.1 Evaluation of Deep Foundations beneath Buildings damaged during the 1994 Northridge Earthquake

The 1994 Northridge earthquake damaged reinforced concrete shear walls of two 16-story apartment buildings in the Los Angeles area. During the ensuing investigation, it was revealed that the piles beneath the buildings were reinforced to only 3.65 m below the pile cap bases. Before rebuilding, it was considered necessary to evaluate the integrity of the foundations. Both towers are founded on 500-mm and 600-mm diameter drilled cast-in-place (CIP) concrete piles. The north west tower is founded on a mixture of older CIP piles, constructed in 1966 and left over from an abandoned project, and newer CIP piles of the same diameters, cast in 1971/2. The older piles were thought to be approximately 15.25 m long. The newer piles were specified on the original foundation plan to have a minimum length of 12.2 m. The same plan gives the position of the pile caps for both towers, as well as the projected pile locations. Pile caps have a nominal thickness of 2 m. The garage basement at pile cap level has a concrete floor slab on grade, 250 mm thick. The soil profile observed at the site comprised a varying thickness of sand and gravel fill under the floor slabs (from 0.6 to 2.1 m), followed by medium dense silty sands to approximately 12.2 m, with dense sands containing cobbles below. The water table at the time of pile construction was 9.1 m below slab level.

The investigation program included three nondestructive testing methods to evaluate the integrity of the existing piles:

a) Ground Penetrating Radar (GPR) to map the borders of the pile caps to be tested below the existing basement slab;
b) Parallel Seismic (PS) testing of selected pile pairs beneath critical pile caps to ascertain pile integrity and stress wave velocity; and
c) Impulse Response tests (IR, also known as Transient Dynamic Response) through the pile caps at selected positions above pile centers to check pile shaft lengths and integrity.

The total program required 5 days on site with a two-person testing team. Information acquired could not have been obtained economically by any other evaluation program. All test methods used are fully described in References 2 and 6.

GPR signal reflections occur at the interfaces between subsurface materials with differing dielectric properties; in this case, the interface between the base of the concrete slab or pile cap and the subgrade below. GPR data are obtained by dragging a suitable radar antenna across the floor and displaying the resultant data on a color video monitor. Vertical interfaces between the 250 mm thick floor slab and the pile caps were located in this manner, in order to position the vertical borings for the PS testing as close as possible to the edge of the pile caps. An example of the GPR image from the floor slab/pile cap interface is given in Figure 3. The mesh reinforcement in the slab can be seen, as well as the more substantial reinforcement in the pile cap.

The Parallel Seismic test was first developed in France for checking the integrity and length of piles where the pile top is not accessible [2,6]. It consists of measuring the propagation velocity of an acoustic wave transmitted through the pile cap of the structure being investigated, received at a number of uniformly spaced points down the side of the pile shaft. A hydrophone is placed in the boring at a fixed position, and the selected test point on the structure is struck with the hammer. The transit time for the stress wave to reach the hydrophone is recorded. The hydrophone position is then changed in equal increments, and the test repeated. The method is illustrated in Figure 4. The cumulated results are plotted as a graph of depth against transit time. A change in the rate of time increase with depth indicates either the base of the element being tested, or a discontinuity in the element. Six different pile caps were examined here, with a boring at each cap to a depth of more than 12.2 m. Refusal of the drilling tool was reached at the bottom of each boring in dense sandy gravel with large cobbles.

Borings for Parallel Seismic tests were fully lined with a PVC casing, 50-mm I.D., sealed at the tube base and well grouted in the surrounding soil. Each boring was vertical and flush against the existing pile cap, spaced equidistantly from the two adjacent piles under test. The restricted headroom and access to the basement area required low headroom drilling, and the casing had to be assembled and installed in 0.9-m segments.

The velocity of the signal, hence the transit time, varies when the wave path includes materials of differing elastic modulus, such as poor quality concrete, or the soil/ rock below the base of the structure. When voids or discontinuities divert the signal, the path length is effectively increased, and the transit time increases accordingly. These variations in transit time are visible as deflections on the time-distance graph, and so the depth of the structure, the uniformity of the concrete and the presence of any defects can be confirmed.

Typical results from the PS testing at this site are given in Figures 5 and 6. The purpose of the PS testing was to measure the average concrete stress wave velocity, to be used in calculating pile lengths from the IR test results. PS tests gave average stress wave velocities of 3,260 m/s.

Impulse Response is a nondestructive test that has been successfully applied to problems where pile integrity is in question [2,7]. The test equipment consists of a hammer equipped with a load cell and a geophone, both of which are linked to a data acquisition unit in a portable computer. Each test is performed by striking the pile head (or pile cap) over the pile axis with the hammer, and measuring the response of the pile to the hammer blow with the geophone, placed approximately 75 to 150 mm inside the pile perimeter. Velocity measured by the geophone divided by the hammer force (v/F) is the mobility of the pile cap and the pile. The depth to reflectors of the downwards-traveling shock wave can be calculated from the mobility plotted as a function of the frequency of the blow over the range 0-2000 Hz.

In the investigation described here, the relatively thick pile cap provided a limited damping effect on the returned signal from the pile, thereby allowing useful information on the pile shaft length and continuity to be obtained. A total of 57 piles were tested by this method, and pile tip length measurements were possible in 49 tests. Using the pile stress wave velocities obtained with the Parallel Seismic tests ensured calibration of pile lengths.
Forty-nine piles tested by the IR method gave definite pile tip responses. The IR test collects raw time-base data for both the hammer force and pile velocity. These test responses can be analyzed in several ways [2]:

(1) Using the velocity-time response to measure the time taken for the stress wave to travel to the pile tip and back (Sonic Echo);
(2) Taking the transfer function of the velocity and force readings to compute the pile mobility spectrum (Mobility);
(3) Using a double Fast Fourier Transform on the velocity time-base data to calculate velocity reflectors; and
(4) Calculating the pile profile by the Impedance Log method.

With the large pile cap in place, the frequency spectrum obtained by the Mobility technique is usually marred by the predominant effect of the pile cap itself, and the response from the pile tip is hidden. This was the case here, and very few pile lengths could be verified by this method.

Some pile lengths were obtained by the Sonic Echo method, with the velocity-time trace exponentially amplified. An example is given in Figure 7. The most successful approach was by the Velocity Reflector method, where the velocity-time trace is transformed twice by the FFT calculation to produce a plot of the most significant stress wave velocity reflectors encountered down the pile shaft. Figure 8 shows an example of such a plot.

The Impedance Log normally does not yield a satisfactory result with such a deep pile cap. For this site, the early part of the velocity-time trace (corresponding to signal reflections down to the pile cap base) was suppressed, in an effort to eliminate the effect of the cap. In most cases, the simulated pile shape was obtained from the Impedance Log.

Figures 9 and 10 show examples of two impedance profiles obtained. Figure 9 indicates a straight-sided shaft of the correct diameter, with the tip reflection at 11.6 m. Figure 10 shows the presence of bulbs on the shaft at 4.25 m and at 8.5 m, with the pile tip at 12.8 m. Of the 49 piles successfully tested, 23 had bulbs at depths varying from 4.25 to 9.5 m, and four had slight section reductions (necks) at between 3.65 and 4.25 m. The pile tip depths varied from pile cap to pile cap, with pile lengths ranging between 11.25 and 17.05 m below the pile cap tops. No test showed any evidence of possible pile shaft breaks at the level of the base of the reinforcing steel. These measured pile tip depths corresponded to the depth of the layer of dense sand with cobbles encountered during drilling for the installation of the Parallel Seismic tubes.

4.2 Concrete Parking Structures Damaged by the Northridge Earthquake

Approximately 100 parking structures were located in areas heavily shaken by the Northridge Earthquake of 1994. These buildings are not unique in the construction materials used, but are unique in the way they are constructed. They are often built using precast, prestressed concrete, with their elements stacked on each other and tied together by either welding embedded steel plates or casting a concrete slab or topping that helps tie the individual members to each other and to the lateral force-resisting system. Their relatively large plan area subjects them to significant temperature changes, thus necessitating minimizing the forces and stresses caused by thermal movements. These factors lead the designer to minimize the number of lateral force resisting systems and to place them as near as possible in the center of the structure. Furthermore, they are generally built with very large ramps that can weaken the lateral force-resisting system, shortening and stiffening the adjacent columns. Finally, there are virtually no partitions, ductwork, ceilings and other elements to help dissipate any earthquake energy input.

The Northridge Earthquake caused significant damage to parking garages. In the Los Angeles area, eight had partial or total collapse, and 20 suffered significant damage. The damage to the south parking garage at the Glendale Civic Center is described here [8].

This garage is a precast, prestressed concrete structure with shear walls providing lateral load resistance. Double tees supported on ledger beams are tied together with a cast-in-place topping slab. Collectors in the topping are used to transmit loads between the precast concrete and the lateral load-resisting shear walls. Three above-grade levels of parking were provided by the precast system. Figure 11 shows the side entrance where partial collapse occurred. Before the earthquake, a third level of precast structure had been present above the entrance. Columns and ledger beams for the bay that collapsed can be seen in the upper center of the photograph. During the earthquake, one column, a ledger beam and all double tees in the bay broke loose from the shear wall and fell to the left (Figure 11). Debris from the collapsed upper level overloaded the two lower floors and caused collapse of the double tees.

Figure 12 shows a connection between shear wall and collector reinforcement in the topping slab. Bars that connected directly into the shear wall were fractured during the earthquake. The wide crack visible along the right face of the shear wall suggests that extensive damage occurred at the face of the latter. Bars at this location must have yielded and may have been fractured. As a result of this damage, the ability to transfer forces between the topping slab and the shear wall was reduced. The lateral load-resisting system consisting of shear walls showed little or no distress following the earthquake. Collapse of one panel of the structure occurred after the capacity of the collector for that panel was exceeded. Damage to collectors in other parts of the top deck diaphragm suggests that extensive inelastic action occurred in the collectors. Inelastic behavior of the collectors resulted in the deck not performing as a rigid diaphragm.

In the event of structures of this type showing little apparent visual damage after an earthquake, it would still be necessary to evaluate the residual rigidity of the topping diaphragm, and any loss of efficiency of the connectors. Some of the NDT techniques described in Table 2 have been used successfully for this purpose, particularly Impact-Echo, Impulse Response and Radar methods. Impulse Radar can evaluate the reinforcing steel in relatively congested areas such as the connectors. Impulse Response can map out significant changes in dynamic stiffness in the topping diaphragm as a result of seismic action, and Impact-Echo can then evaluate the reasons for any loss of stiffness.

5. CONCLUSIONS

The case histories presented in this article illustrate the technical and economic advantages of incorporating modern NDT techniques in evaluation programs for earthquake-damaged structures.

In the case of the Northridge Earthquake deep foundation studies, the nondestructive testing program described here proved to be the most practical approach available for the evaluation of the integrity and the length of critical piles under the two buildings, with a minimum amount of disruption and cost. The confidence obtained in the Parallel Seismic test results allowed full subsequent use of the Impulse Response test for the remaining piles. The parking garage in the basement remained in service throughout the field-testing.

Damage to parking structures as described in the second case history can be quantified using an approach relying mainly on visual observations and nondestructive testing, for full and economic coverage of the problem.

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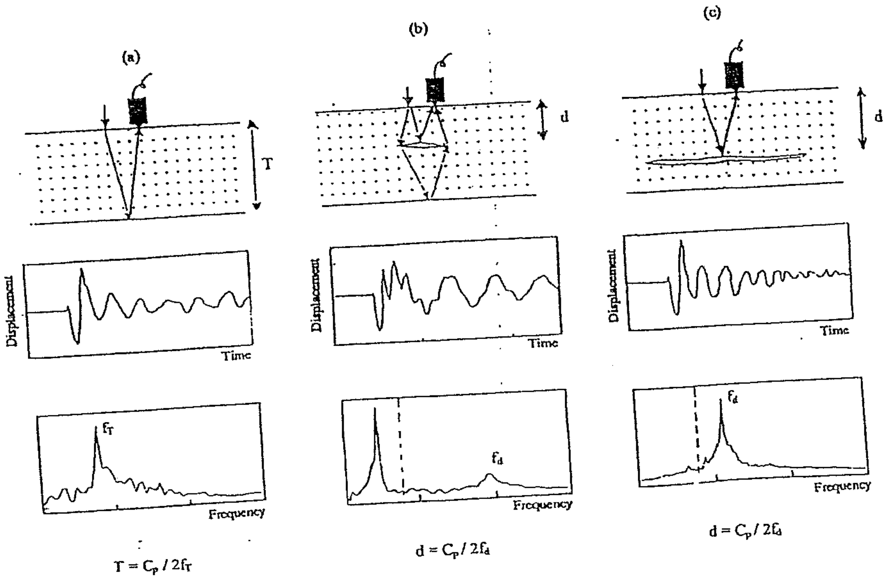
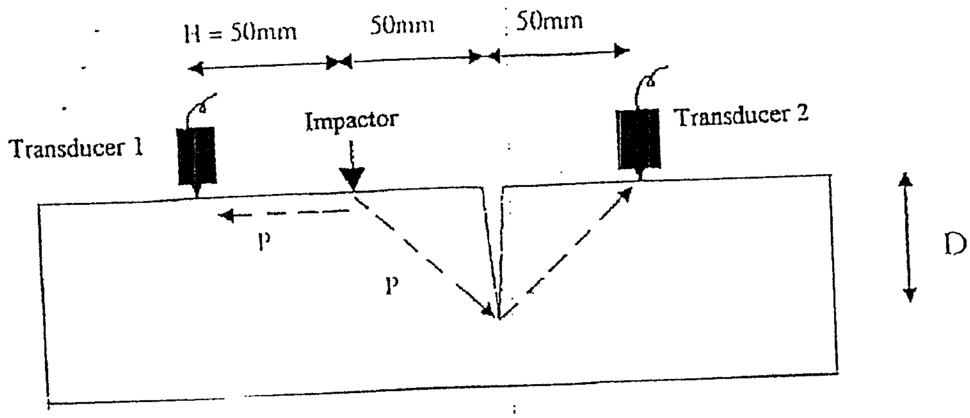
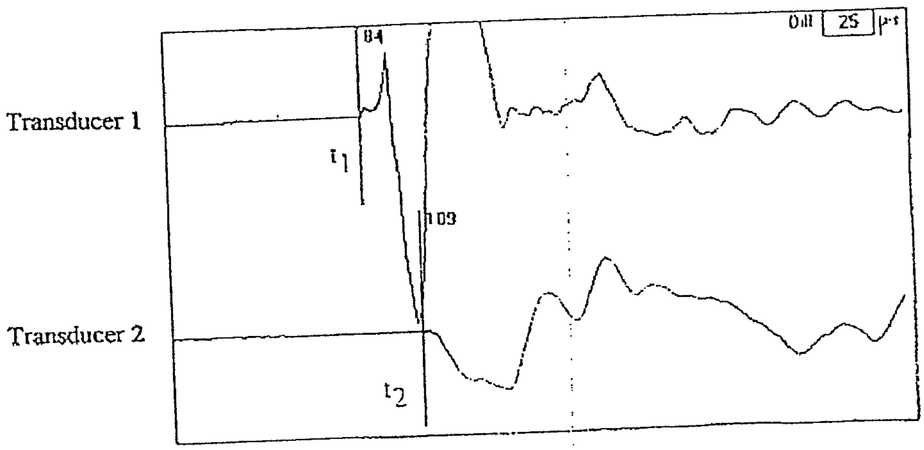


Figure 1. Impact-Echo; Principle for measurement of (a) Plate Thickness (b) & (c) Location of Flaws in Plate Elements



$$D = \sqrt{\frac{(C_p \cdot (t_2 - t_1 + H/C_p))^2}{4} - H^2}$$

Example:



For $C_p = 3900$ m/s, $t_2 - t_1 = 25$ microseconds and H=50 mm, the depth D of the crack is:

$$D = \sqrt{\frac{(3900\text{m/s}(25 \cdot 10^{-6}\text{s} + 0.05\text{m}/3900\text{m/s}))^2}{4} - (0.05\text{m})^2} = 54 \text{ mm}$$

Figure 2. Impact-Echo; Testing Depth of Surface Crack

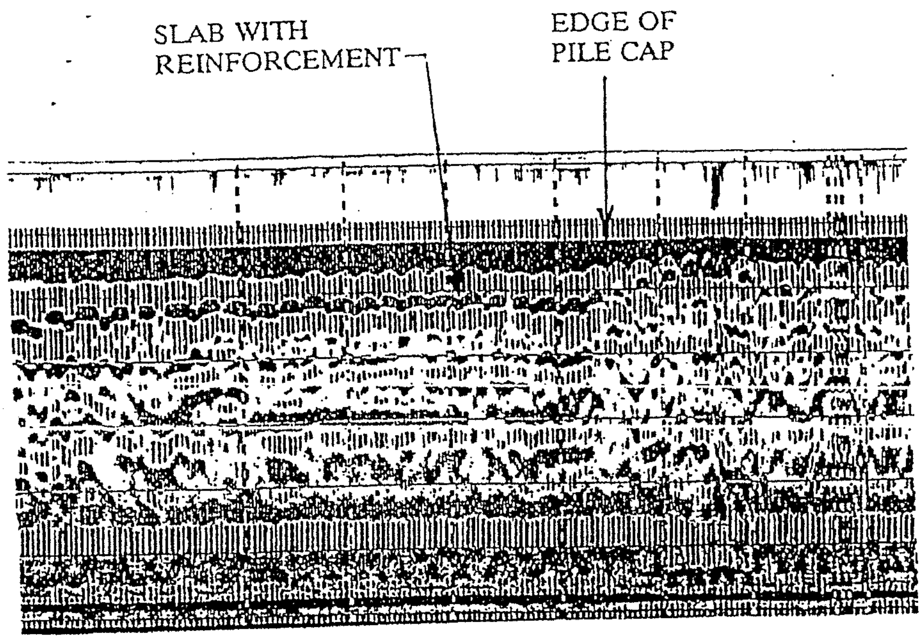


Figure 3. Typical ground Penetrating Radar Profile

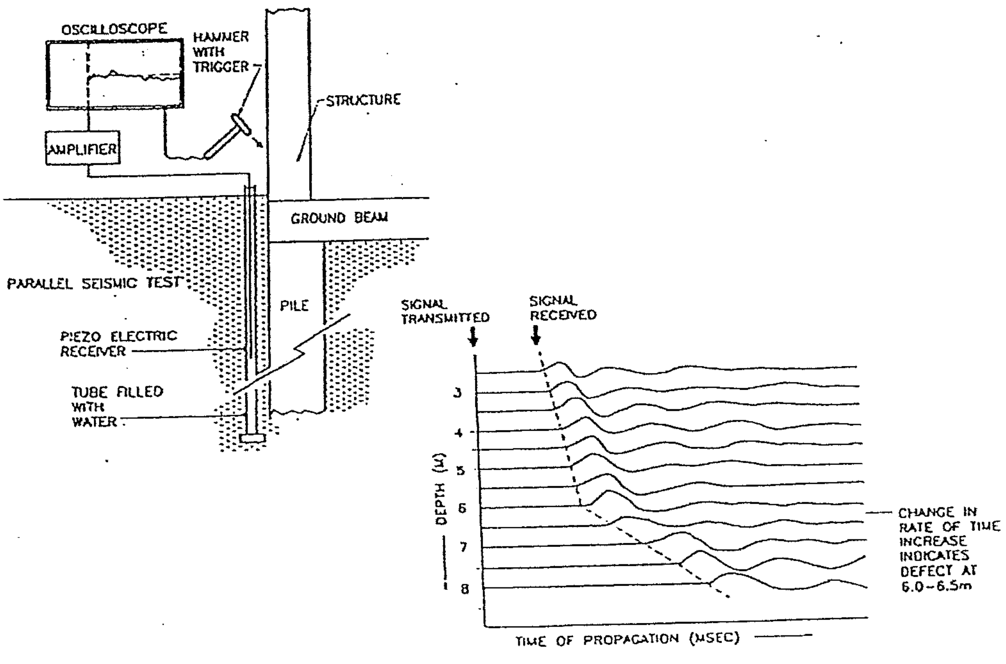


Figure 4. Principle of Parallel Seismic Test

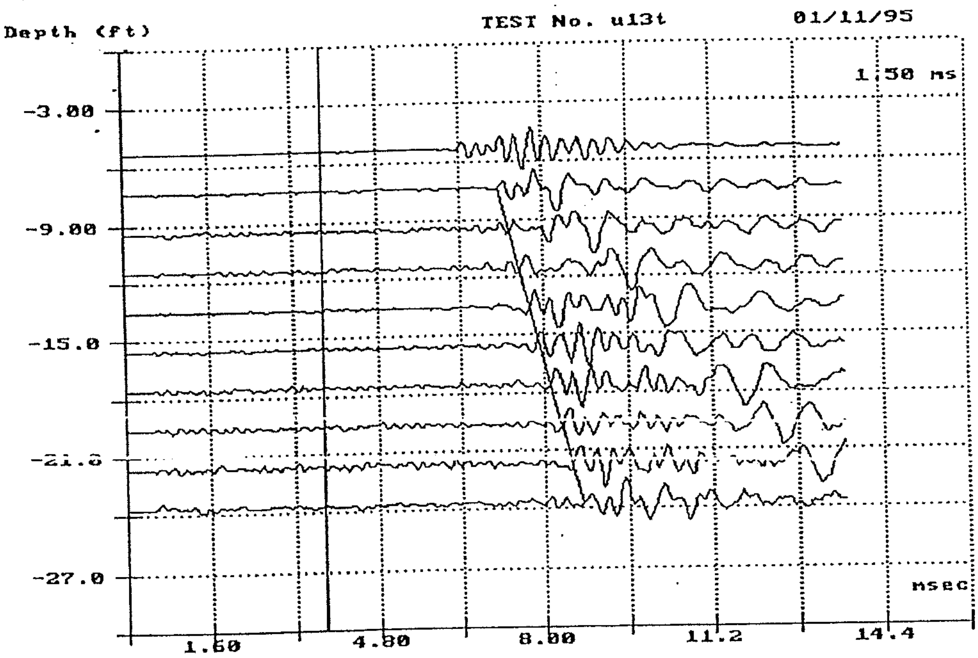


Figure 5. Parallel Seismic Test Profile – Pile Stress Wave

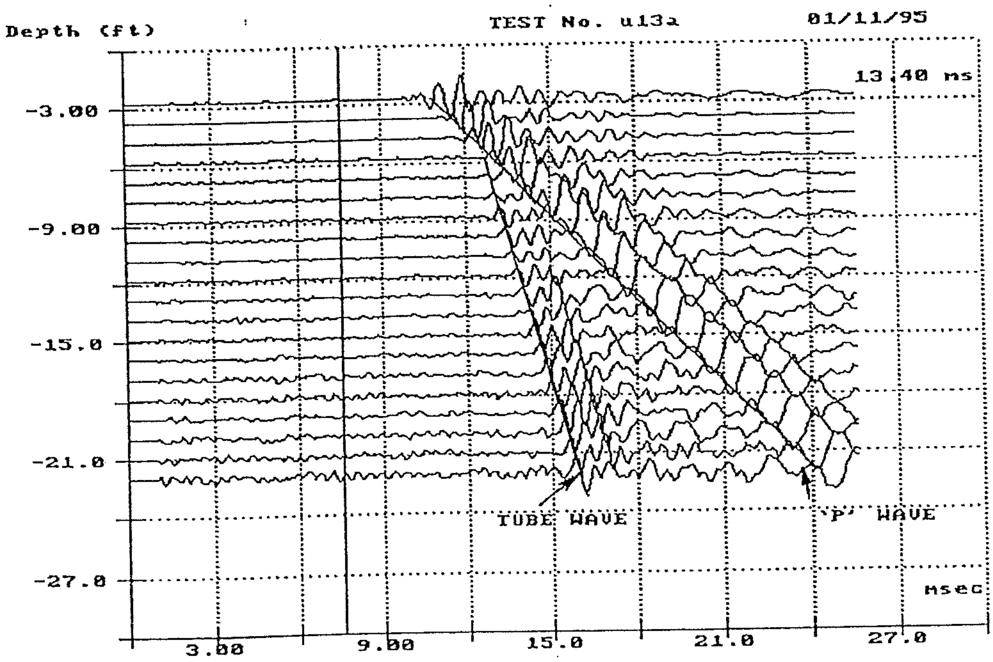


Figure 6. Parallel Seismic Test Profile – Ground 'P' wave

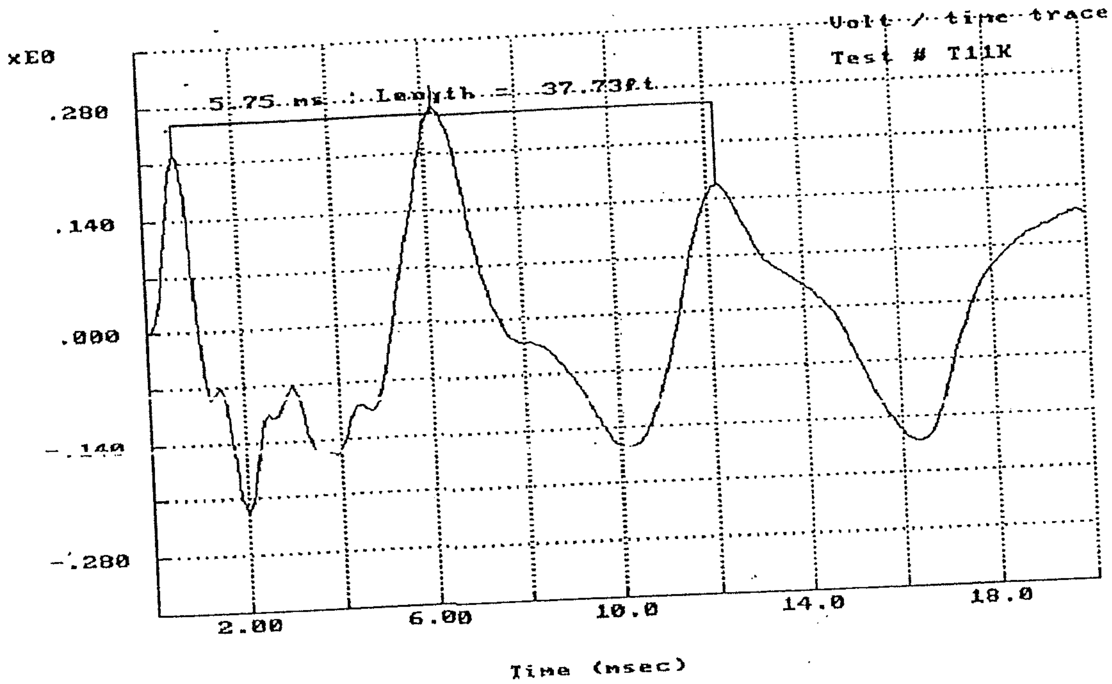


Figure 7. Amplified Sonic Echo velocity-time Trace

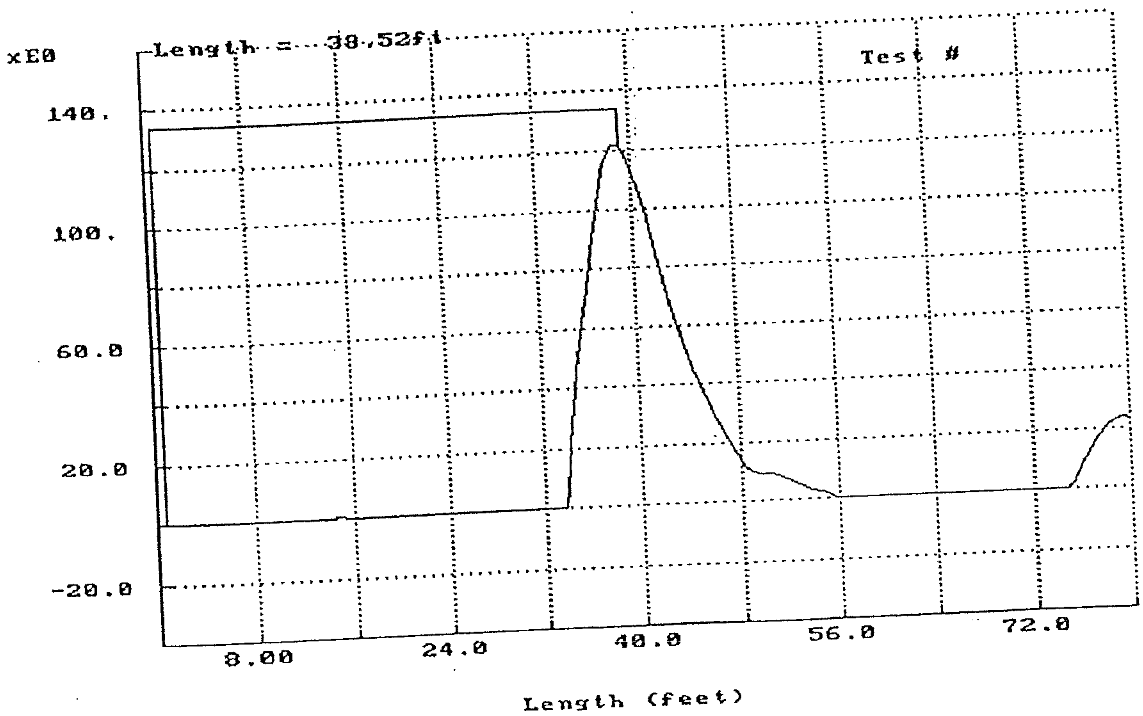


Figure 8. Velocity Reflector Trace

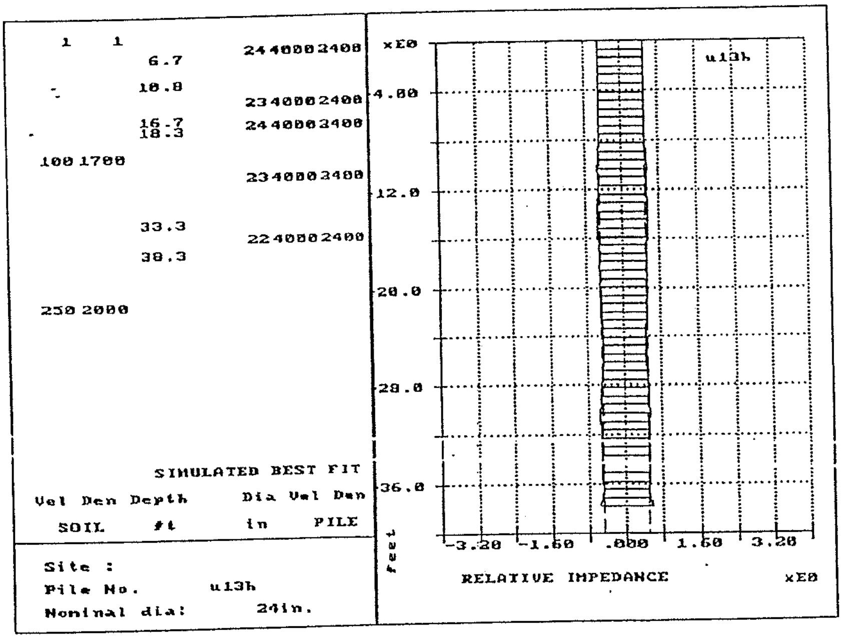


Figure 9. Impedance Log Profile – Straight-sided Shaft

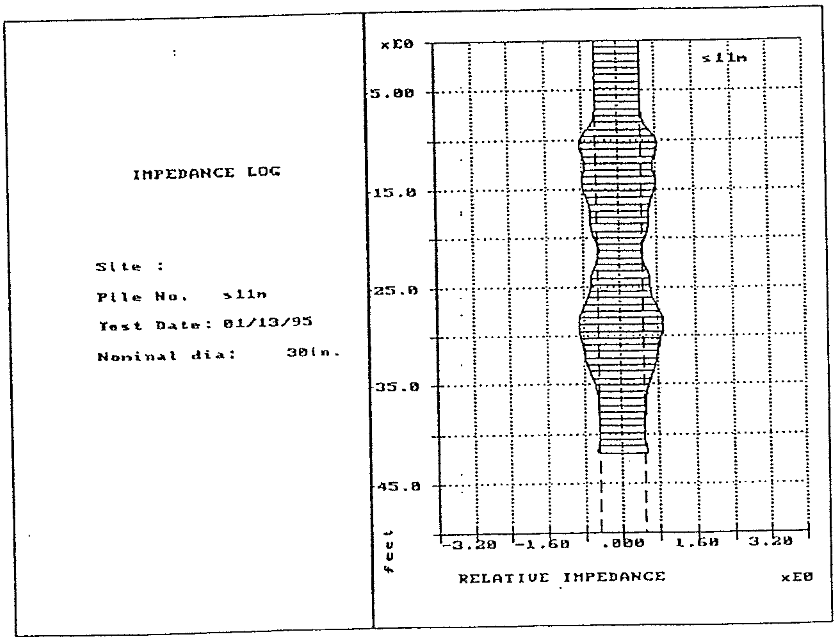


Figure 10. Impedance Log Profile – Shaft with Bulbs

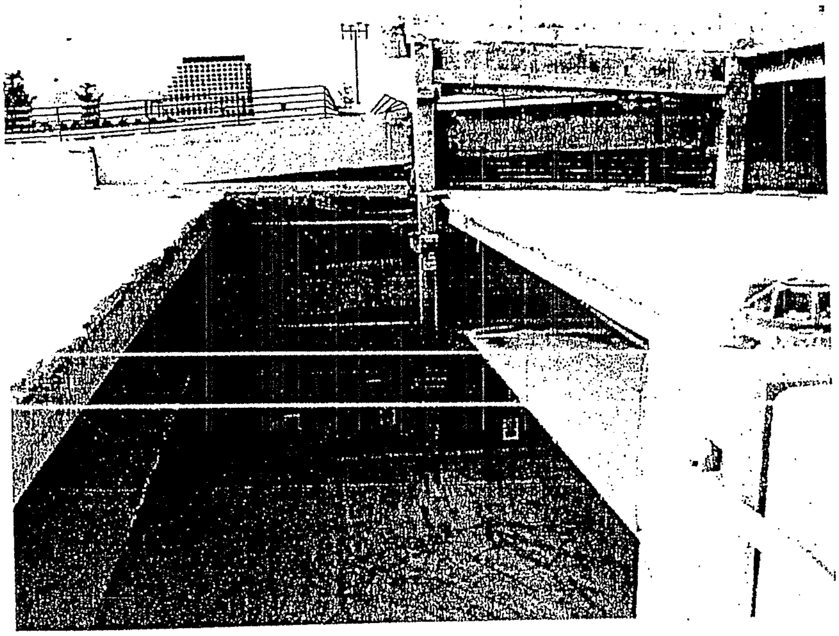


Figure 11. Collapsed third above-grade level

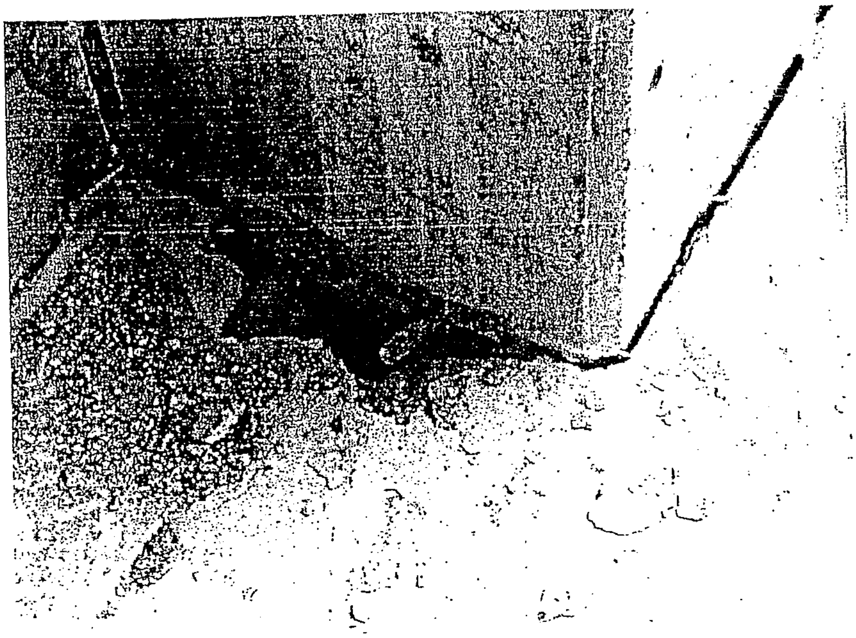


Figure 12. Broken collector reinforcement at face of shear wall