EDWARDS UNDERGROUND
WATER DISTRICT

Report 92-01

Blasting Effects
on Engineered Structures

BLASTING EFFECTS ON ENGINEERED STRUCTURES

A Technical Investigation

for the

Edwards Underground Water District

by the

Center for Water Research

University of Texas at San Antonio

August, 1992

Executive Summary

Blasting Effects on Engineered Structures

The investigation of the effects of blasting was commissioned by the Edwards Underground Water District (EUWD) to determine how blasting degrades the integrity of underground structures and what criteria might be effective to limit facility degradation. The study successfully achieved both objectives, and the major findings are as follows;

(1) None of the structure types investigated in the field (wells, pipes and tanks) suffered damage causing loss of their contents or loss of function when production charges were detonated at distance greater or equal to 10 feet. All of the structures in the study were in good condition and installed according to city and state specifications. More conservative criteria should be applied to structures and systems that have not been constructed according to specifications and/or have become more fragile with age. Also, the total size of the blast and the delay sequence must be considered.

(2) Well-behaved relationships between the amplitude of blast response at a given location and the size of the energy source (velocity response ratio, VRR) were obtained. These relationships indicate that the vibration response to production blasting at selected locations may be predicted with reasonable accuracy from the measured site response to small (calibration) blasts.

(3) Variations of blast response amplitude with depth (normalized velocity response, NVR) for the target structures were also well behaved. In general, amplitude decreases with depth. When the charge is detonated close to the structure, the amplitude may be relatively constant with depth, or may increase with depth for large, close events.

The longitudinal (radial) component of motion is most likely to display the latter behavior, probably due to improved energy transmission in saturated ground.

(4) The relationships between source energy and response amplitude, and the relationships between amplitude and depth may be combined in a response prediction procedure. First the site response to a small charge (eg. 1/2 lb. of dynamite) is measured at points of interest. Next the ground surface response to a production blast is determined from the low-energy response and the energy/amplitude relationship, VRR. Finally the subsurface response in and around the structure is predicted from the surface response and the relationship between amplitude and depth, NVR. Details of the development and application of the procedure are given in section 7 of the report.

The prediction techniques developed in the study show promise for application to vibration control during production blasting. Application of the method in construction will provide an opportunity to collect a reservoir of data which can be used to improve the accuracy and precision of site response predictions. Reducing uncertainty in blast operations will help the excavation contractor to complete his job more safely and efficiently, and will reduce the risk of aquifer contamination by limiting damage to underground structures.

The investigators acknowledge the generous support of the Edwards Underground Water District for the work completed under this contract, and look forward to providing future assistance to the District in the development of solutions to related technical problems.

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1.0 INTRODUCTION

Past experience indicates that ground vibrations from quarry and construction operations in the Edwards Aquifer Recharge Zone (ERZ) and the surrounding areas are capable of causing significant damage to constructed facilities including buildings, storage tanks, piping systems, reservoirs and wells. A potential for damage to natural underground water conduits and springs, and to natural recharge features, including caves and sinkholes, also exists. The destructive potential of blast-induced vibrations to subsurface facilities is particularly severe since the effects of the vibrations are not easy to detect, and when the damage is discovered, remedial action may be difficult or impossible.

Damage potential is particularly acute on the ERZ and the adjacent drainage area in north and northwest Bexar County. The highest growth rate in the county is projected for these two areas, based on construction of new highways, subdivisions and entertainment facilities. With little or no soil cover in most of the region, hydraulic structures are usually installed by blasting, with potential for damage to existing structures. Blasting also fractures rock around the excavation, and often exposes recharge features, increasing the potential for contamination to the aquifer. Quarry blasting, highway excavation and other blasting activities related to infrastructure development also increase the level of vibration loads to which subsurface structures are subjected. No blasting regulations exist in the unincorporated areas of the County, and blasting is under-regulated within the city limits. As a result, there is no control on the size and placement of charges in excavation operations. The ability of City and County agencies to inspect subsurface facilities is extremely limited because of a lack of funding, personnel and equipment.

The Edwards Underground Water District (EUWD) and the Center for

Water Research (CWR) at the University of Texas at San Antonio (UTSA) have recognized the importance of developing techniques for dealing effectively with the above problems. Methods for predicting ground-motion amplitude and frequency at specific locations, and damage potential as a function of charge size, delay pattern, local geologic conditions and distance from the blast have been developed to suit the needs local blasting practice. These procedures determine recommended charge size and distance of blasting from engineered structures based on vibration tests conducted at the site. The solution, which is the result of more than two years of extensive research, provides an integrated approach to a previously unsolved problem. Work included research, field testing and monitoring, analysis and synthesis of operational criteria. The results of the work include a narrative of project activities, reduction and analysis of the field data and recommendations for development of safe and efficient blasting practice on the ERZ.

The preparation of criteria for blasting practice and the eventual application of the criteria in this region by the EUWD would contribute significantly to reducing the risk of an unexpected degradation of ground-water quality. The investigators acknowledge the support of the EUWD in this important and timely effort, and look forward to the implementation of the project recommendations as a credit to the District and the many community leaders who have been instrumental in its success.

2.0 BACKGROUND

Blasting has historically been the method of choice for excavating large volumes of rock in the crystalline carbonate units of the Edwards Group and the Glen Rose Formation in south-central Texas. Non-explosive excavation techniques do exist. None, however, is feasible for operations which require moving large quantities of rock economically.

2.1 Blast Vibration Sources

In this region the most common applications of blasting techniques are in quarry operations, construction and utility excavation. The great amount of activity in quarry operations to the west, north and northeast of San Antonio along the Balcones Escarpment reflects the tremendous economic importance of limestone of the Edwards Group as a source of high purity calcium carbonate for industrial and chemical purposes, agricultural applications, and construction uses in aggregate, Portland cement and soil stabilization. Large scale rock excavation for highways, dams, surface drainage projects, tunnels and utility trenches in areas of relatively low population density is usually accomplished by blasting. Utility trenches in very high density population areas are generally excavated by techniques which do not involve explosives (rock saws, pneumatic hammers, etc.). The relatively high cost of these methods indicates a need for the development of efficient excavation techniques with acceptable vibration levels.

2.2 Effects on Structures

With the use of explosives there is an inherent potential for significant damage to both man-made structures and geologic features. Vulnerable above-ground structures include irrigation canals, reservoirs, dams, and facilities for the production, storage, and transportation of hazardous solid and liquid materials. Pipe connections between surface structures and subsurface pipes or tanks are particularly vulnerable to the differential displacement which occurs between the blast-excited ground surface and above ground structures. The effects of blasting on and damage criteria for above-ground structures have been examined by several researchers in the past (References 1 and 2). Complete information regarding the effect of relative motion between the ground surface and surface structures is, however, not available.

Sub-surface structures are of particular importance from the standpoint of potential for blast-induced vibrational damage. The structural integrity of tanks, lift stations, pipes and wells is often difficult to accurately ascertain before blasting. The initial ground-water flow conditions and structural integrity of caves, sinkholes, subsurface solution channels are equally difficult to assess. Complete standards for acceptable vibration levels on man-made structures and geologic features have not yet been developed, and these structures are often at risk during blasting operations. The effects of blasting on shallow subsurface structures and geologic features are a critical component of the investigation because of the possible negative impacts of ground-water quality and quantity.

2.3 Synthesis of the Solution

The ideal method for limiting the adverse effects of blasting on structures and subsurface features is by separating the blast sources from the potential damage sites by great distances. Growth trends in metropolitan San Antonio and surrounding areas are in sharp contrast to these ideal circumstances. San Antonio is expanding rapidly to the north and west, towards the areas which contain the highest levels of quarry and construction activity. New quarries and highways are planned and existing quarries and highways are being expanded while residential and commercial development in the same areas and in close proximity to some quarry operations is increasing. The development of dependable methods for predicting blast-induced vibrations and criteria for acceptable ground motion levels will enhance our ability to protect man-made structures and natural features from the potential adverse effects of blasting related to quarry operations and construction activities, and thereby safeguard the Edwards aquifer from degradation by effluent and hazardous materials.

3.0 PROJECT ORGANIZATION

3.1 Personnel

The principle investigators on the project were Drs. Weldon Hammond and Geoffrey Blaney of the U.T.S.A. Center for Water Research (CWR). Dr. Hammond has an extensive background in local geology and geohydrology, and is the Director of the Center for Water Research. Dr. Blaney is a geotechnical engineer specializing in ground vibration analysis and environmental engineering, and is a Research Scientist at the CWR. The principle investigators were assisted by the technical staff of the CWR. Construction subcontractors worked under the direct supervision of the principal investigators.

3.2 Organization of the Work

From an operational standpoint, the work was divided into three phases;

Phase I - Organization; site and materials studies and background research

Phase II - Execution; Response predictor development and field testing

Phase III - Reporting; data reduction, report preparation and presentation.

Project Tasks

The project work included both analysis and testing, and was divided into the following six major technical activities;

1) Site and construction materials acquisition study.
2) Background investigation and literature search.
3) Development of simple predictors of ground motion and damage based on controling parameters for the blast-rock-structure system.
4) Full-scale blast loading of representative structures to determine

dynamic response effects.

5) Incorporation of testing results into practical ground motion prediction techniques and working criteria for construction applications.

6) Testing of predictors and criteria under working conditions in the field and instruction of EUWD staff in criteria application.

All phases of the work were closely coordinated with EUWD staff. Progress reports, meetings and site visits kept the District representatives abreast of investigation activities. The EUWD has had opportunities to make significant input to the organization and execution of the project. Technical input by EUWD staff was very helpful in identifying the specific types of engineered structures and geologic structures of interest and their age and spacial distribution in the study area. Assistance by EUWD staff in identifying and obtaining access to field test sites and obtaining materials for the field tests was also invaluable. Diane Poteet was particularly helpful in this respect. Staff members had numerous opportunities to collaborate with investigators during the project, with the object of achieving significant technology transfer to the EUWD in the areas of field data acquisition, wave propagation analysis, geophysics and application of criteria.

4.0 PRELIMINARY PROJECT ACTIVITIES

4.1 Site and Construction Materials Study

The first major task in the investigation was identification of appropriate sites for the study and obtaining access to them. The following sites and project applications were identified;

1) **San Antonio River Authority (SARA) Dam Number 10** (near Redland Road and Jones Maltsberger). Blast response of a 36-inch-diameter buried concrete sewer pipe.

2) **Highway 1604 at Judson Road.** Blast response of a cased water well.

3) **Highway 211** between Highway 471 (Bexar/Medina County line) and Highway 16 (near San Geronamo). Blast response of the Glen Rose formation and the Edwards Group.

4) **Northwestern Quarry** (Judson Road north of Highway 1604). Ground response to quarry blasting.

5) **Stone Oak Development**, Wild Sunday Farm (formerly Champions Equestrian Center), north of Stone Oak Parkway. Detailed study of the response of pipes, tanks and natural recharge features to controlled blasting.

Access to all of the above sites was arranged by the EUWD staff. The District staff also helped to coordinate daily activities at the SARA and Highway 211 sites. The sites are sufficiently diverse geologically to represent conditions encountered throughout the ERZ and the drainage area.

During the site-selection phase of the work, a survey of construction methods and materials was conducted. Specifications and manufacturers' product descriptions were collected for PVC and concrete pipe, and steel and fiberglass tanks. Local installation practice was also documented through site visits and discussions with local contractors.

In preparation for field data aquisition, a 32-channel microcomputer-based data acquisition and analysis system was designed and tested. The system proved to be cost-effective compared with comparable proprietary systems on the market, and the in-house design fit the needs of the project well.

4.2 Background Investigation

The background investigation began with a literature search which focused primarily on government reports and regulations that define damage criteria for conventional surface structures and recommend simple equations for predicting ground surface motion amplitudes. Technical publications describing ground motions and measurements of damage from blasting were reviewed for information pertinent to the project. Specifications and regulations for blast vibration control were also documented. Data from the publications was incorporated into the development of the initial criteria. Dialogues with blasting contractors provided a practical local background for the investigation.

4.3 Response Predictor Development

Numerical models for predicting the variation of ground surface response amplitude with distance from the explosive source are available in the literature today. A simple response equation for ground surface particle velocity (Reference 2) would take the following form:

$$v = H \left\{ \frac{D}{W^{a}} \right\}^{b} \tag{4.1}$$

in which: v = particle velocity

H = measured (or estimated) particle velocity corresponding to $(D/W)^{1/2} = 1.0$

D = distance from charge

W = maximum charge weight per delay

a = charge weight regression exponent

b = regression exponent or slope of the velocity response graph.

More sophisticated models, based on the principles of response spectra analysis (Reference 1) which are widely used in earthquake engineering, are also available. The existing models served as a starting point of development of methods for prediction of wave propagation characteristics in crystalline carbonate rocks encountered in the study area. The results of the National Science Foundation (NSF) - funded wave propagation studies conducted at the CWR (Reference 3), in which unique data acquisition and reduction techniques were developed, also contributed to the development of these models.

Current blasting practice relies on non-site-specific numerical relationships, (eg. Equation 4.1) to predict vibration levels associated with a given charge weight. Application of these equations does not always provide consistent results. A more precise method, relying on site calibration for response to low level blast exitation, was formulated in this task. The completion of this work is discussed in the following section, and is the major contribution of the project work.

The response prediction techniques were developed for PVC and reinforced concrete sewer pipes and associated manholes, steel and fiberglass underground storage tanks and cased water wells. All of these structures were tested at full scale in the field.

5.0 FIELD TESTING

In the field testing program, full-scale representative engineered and natural structures were subjected to blast loadings of varying duration and intensity. Engineered structures were instrumented with velocity transducers and subjected to a complete visual inspection before blast testing. 32 channels of ground motion were recorded from sensors placed at locations adjacent to the charge, between the charge and the structure, on the ground adjacent to the structure, and at selected points on or in the structure for each blast. Structures were carefully inspected and tested after blast loading for rigid-body displacement, fatigue and failure.

5.1 Data Acquisition System

The data was acquired on a PC-based system assembled by CWR staff from state-of-the-art hardware and software components (Appendix C). The velocity sensors (geophones) generated a time-varying voltage linear with the ground motion velocity (Appendix B). This signal was carried over twelve-conductor-pair cables to the preconditioning unit, where each channel of analog signal was amplified at a user-selected descrete level between 1 and 1000 (Figure 5.1). The conditioned signal then passed to the acquisition boards, where data from each channel was multiplexed and digitized.

The sampling rate for each channel was 1024 samples per second, allowing signal discrimination to a frequency of 500 Hz. The cut-off frequency was more than adequate, because the geophones were only calibrated by the manufacturer to 400 Hz. Most of the ground motion energy was transmitted at frequencies below 200 Hz. The acquisition board functions were controled by dedicated software. The digital data was stored on a 160 megabyte hard disk, then demultiplexed and stored as individual channels of digital data. The data was reviewed in the field to assure proper function of the acquisition

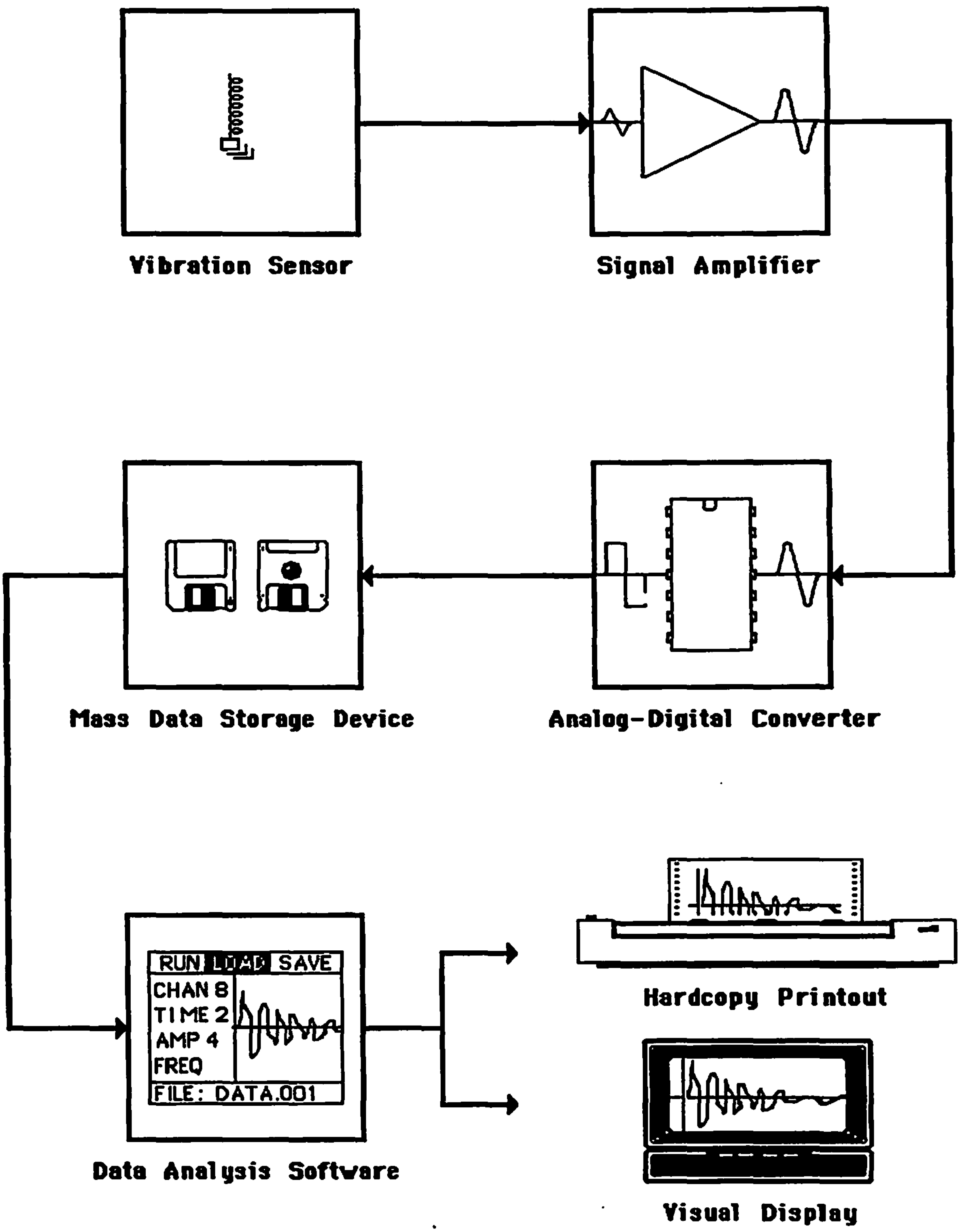


Figure 5.1 - Data Acquisition System Functions

system and to confirm proper amplifier gain settings for each channel. Work completed for each testing site is described in the following sections.

5.2. San Antonio River Authority Dam Number 10 Testing

The geology at this site (Figure 5.2) is an excellent example of the geologic conditions encountered on the ERZ near its southern boundary, where the upper-most members of the Edwards Group are exposed (Figure 5.3). The Del Rio Clay, which is overlies the Edwards Group, is exposed in a stream bed directly south of the site, indicating faulting in the area. The walls of the stream channel in which this flood control structure is built contain numerous solution channels and caves. Open and clay-filled caves and solution channels were encountered in the preliminary geotechnical investigation for the project, and during the excavations for the dam keyway. These geological conditions present the greatest risk for contamination from loss of sewer outfall integrity under blast loading, and therefore the site was an appropriate starting point for the field investigations.

The response of an existing 36-inch-diameter reinforced concrete sewer pipe to production blasting for the east dam abutment was recorded. The outfall was not in service during the vibration measurements, however, a small amount of flow from infiltration was observed. A 200-foot-long section of pipe runs perpendicular to the dam axis between manholes located on the upstream and downstream sides of the dam. The pipe was laid in a trench excavated in the limestone by blasting, and is 27 feet beneath the ground surface at the manhole to the north of the dam. Instruments were placed on the ground between the east abutment and the outfall, and at three stations in the outfall; adjacent to the base of the manhole on the pipe invert, at the quarter point between manholes and at the half-way point between manholes. The highly-variable rock quality and karst features present in

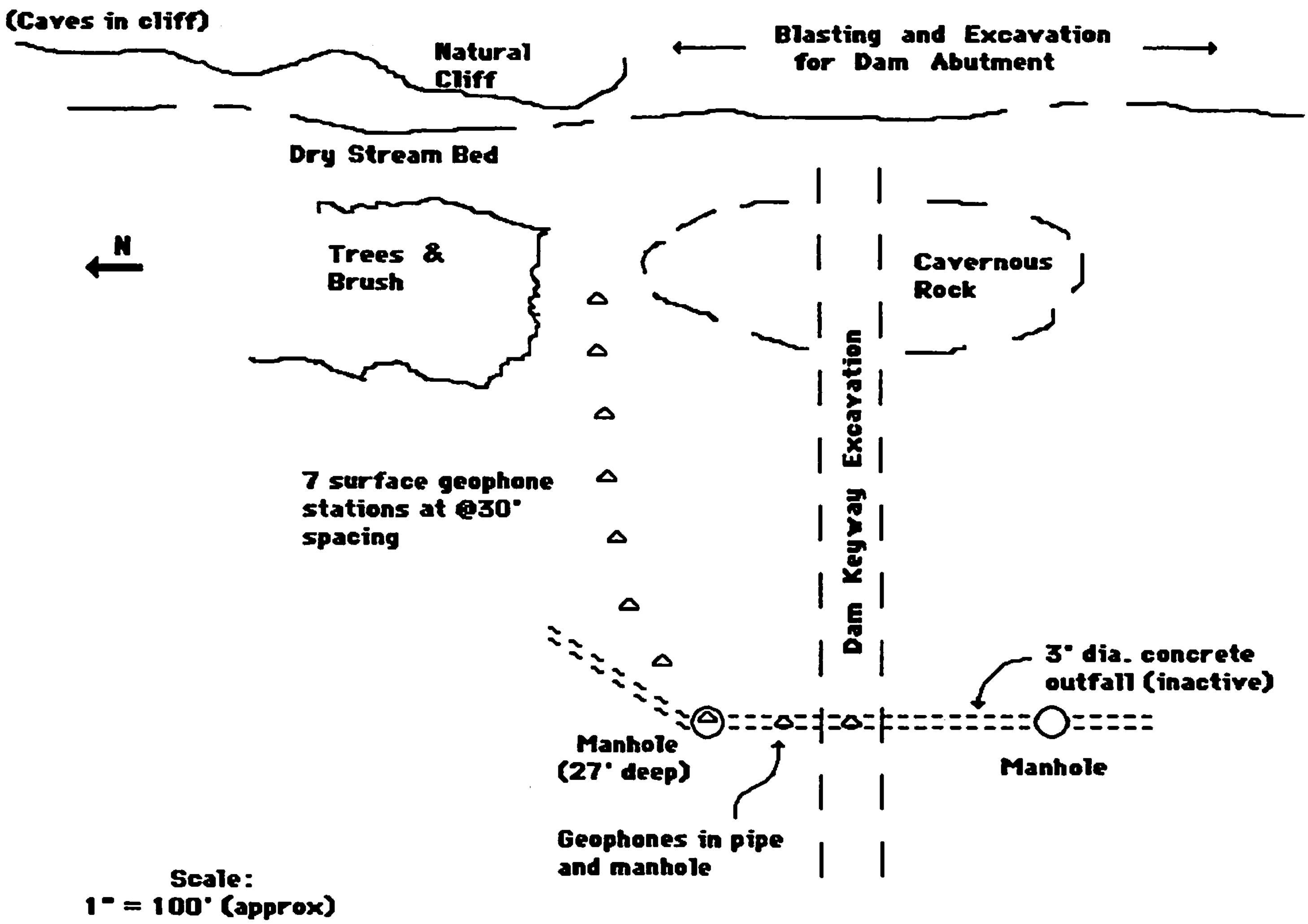


Figure 5.2 - San Antonio River Authority Dam Number 10 Site

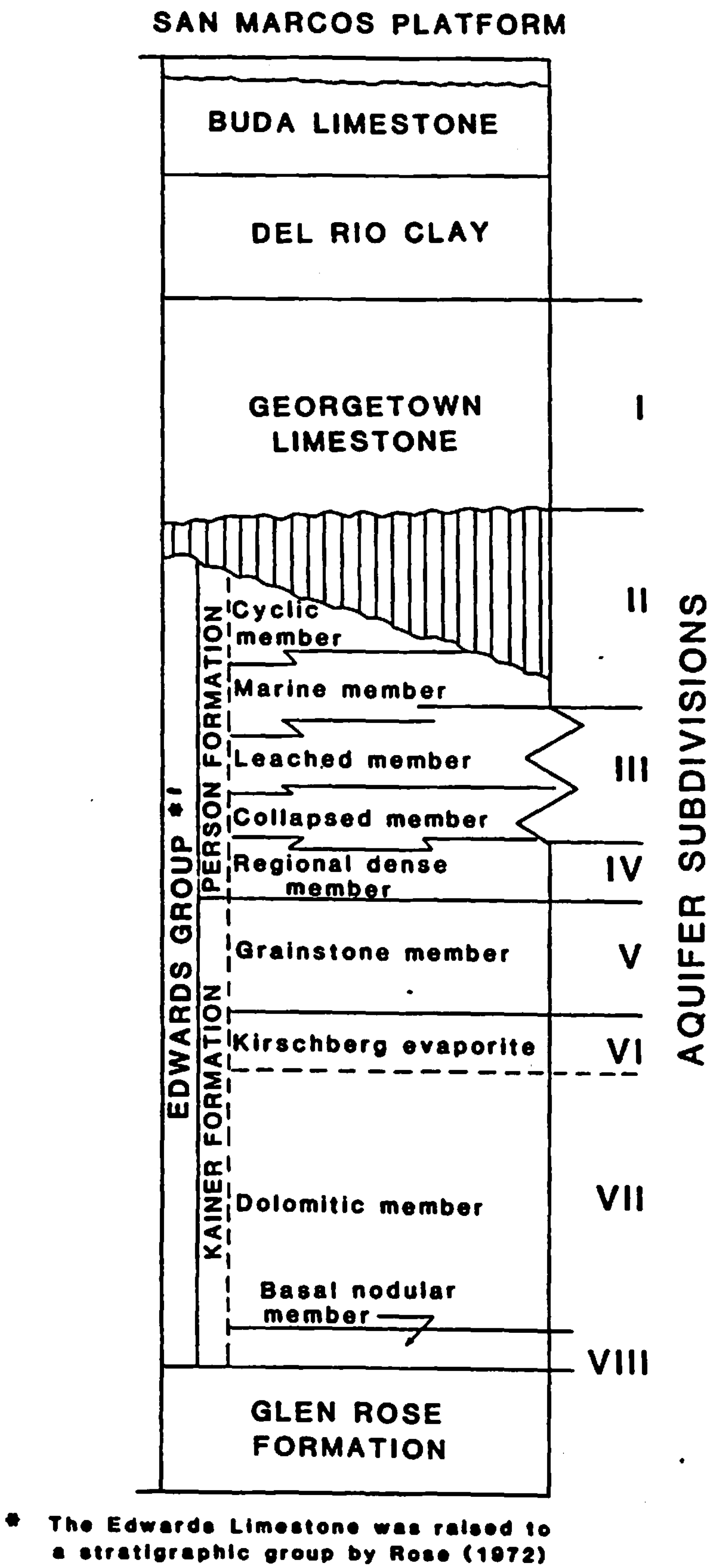


Figure 5.3 - Geologic Column for the Edwards Group and Related Formation

the dry bed of the Salado Creek are reflected in the variability of recorded response at different locations on the ground, and the variation in pipe response with change in the location of the blast.

5.3 Highway 1604 at Judson Road Testing

Blast excavation to grade for the access road at the northwest corner of the intersection (Figure 5.4) passed within 5 feet of an existing cased well which penetrated the Buda limestone into the upper Edwards Group. Three-dimensional and two-dimensional instrument packages were placed 5 feet and 27 feet below the ground surface, respectively, and held in place with a pneumatic locking system. Ground and well casing response to numerous blasts, each with a different charge weight, delay sequence, geometry and distance to the well were recorded.

Array sizes ranged from 10 holes to 52 holes with charge weights between 225 lb. and 1050 lb. Hole depths were between 7 and 14 feet. Most of the arrays were symmetric with three lines of holes parallel to the excavation face on the west side of the site. Blasts would generally open with the center line of holes followed by the line closest to the face, and finishing with the line furthest from the face. Within each line, the blast would open at the center of the line. The delay between each line was 17 milliseconds and the delay between holes in a line was 25 milliseconds. Water was encountered in some of the holes due to down-slope seepage, in which case 1- inch-diameter by 4-inch-long 75% boosters were placed at the bottom of the hole to assure complete detonation.

The pattern of the ground vibration response was controlled primarily by the blast geometry and delays, and not by the ground conditions, for most of the blasts because the blasts were relatively large, and the distance to the instruments relatively small. The blasts recorded at this site presented an

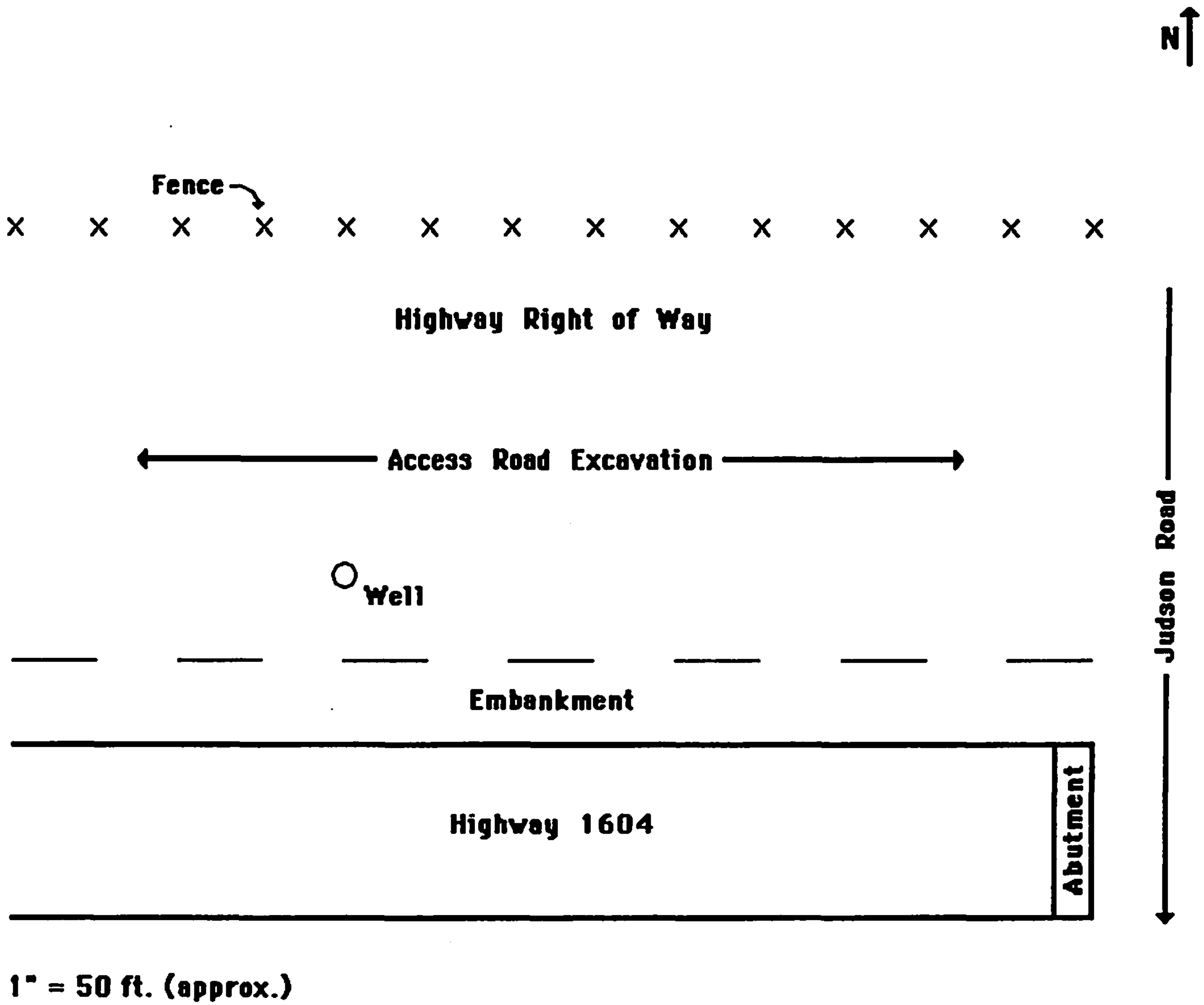


Figure 5.4 - Highway 1604 at Judson Road Site Map

excellent opportunity to gauge the considerable variability in the vibration amplitude, duration and frequency that might be anticipated in normal constuction operations.

5.4 Highway 211 Testing

This site (Figure 5.5) is of particular interest because the upper part of the Glen Rose formation and the lower Edwards Group are exposed in a wide north-south cut. Ground response to blasting for cut and fill operations was recorded to characaterize the relationship between rock type, rock quality and vibration transmission.

5.5 Stone Oak Development Testing

A considerable amount of time was required to secure permission for the use of this site. It proved, however, to be an excellent location, because the varied geology was representative of sensitive drainage areas on the ERZ, and because it was both convenient to the UTSA campus (15-minute drive) and separated from residential developments by rolling hills. It was here that the majority of the project work was conducted. Without access to this site, the project productivity would have been severely restricted.

5.5.1 Site Investigation

This site was investigated extensively before blasting to determine the ground conditions along the centerline of the pipe trench and in the surrounding area. Electromagnetic and surface wave geophysical surveys were conducted to determine the effectiveness of these methods in the local terrain. The results of these tests were correlated with the logs of borings and excavations along the pipeline allignment and adjacent to it (Figure 5.6, Appendix A). The anomalities indicated by the geophysical surveys were later found to be associated with open and clay-filled voids in the limestone.

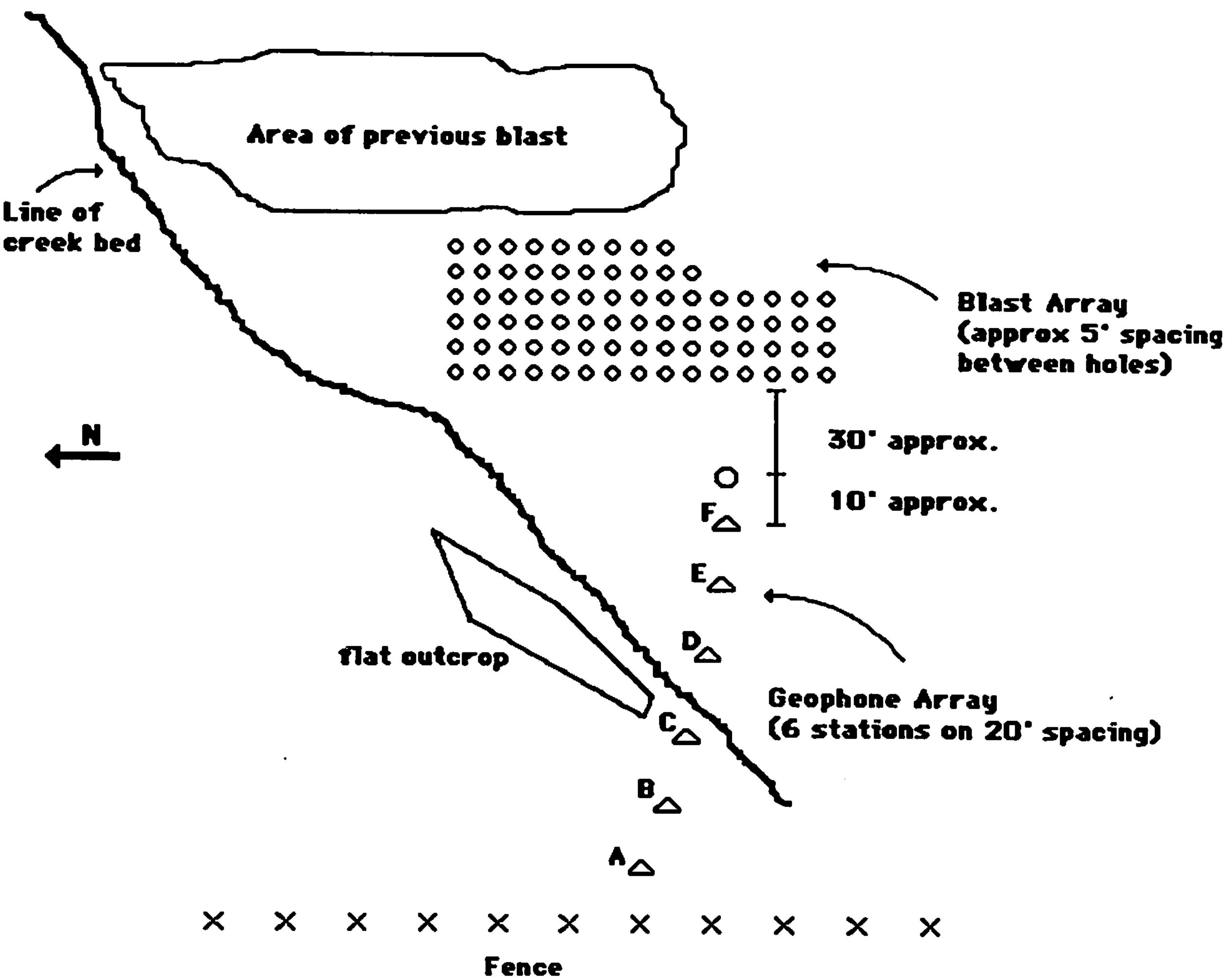


Figure 5.5 - Highway 211 Blast Site Map

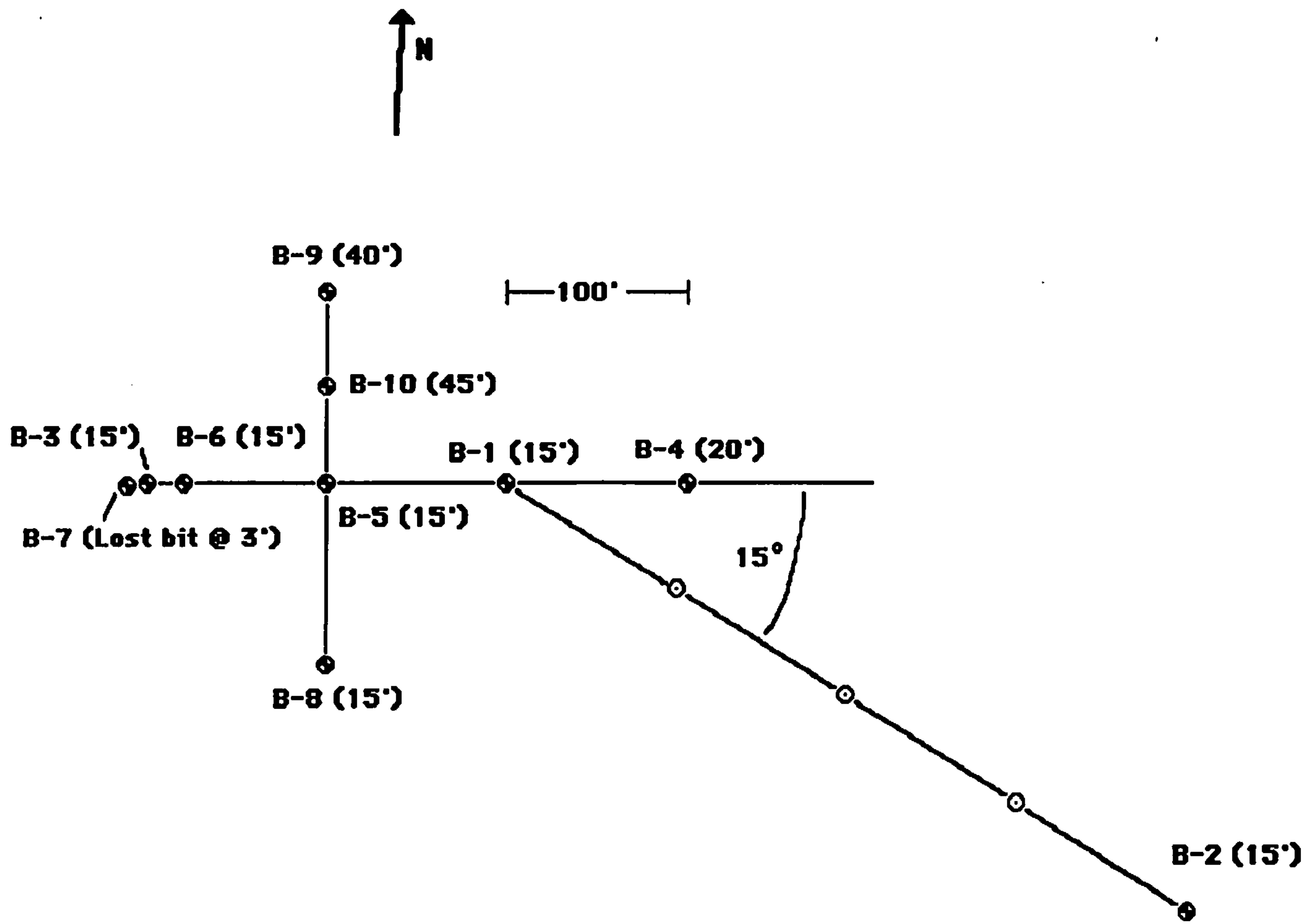


Figure 5.6 - Stone Oak Field Investigation Map

Like the SARA Dam Number 10 site, the geologic conditions at this site (Figure 5.7) reflect the worst-case situation with respect to possible infiltration from damaged pipes or tanks. In this case, however, the geologic formation is the Basal Nodular member of the Kainer formation, which lies at the base of the Edwards group over the Glen Rose formation (Figure 5.3). The Kainer formation at the site location is at least 45 feet thick (Appendix A) and is characterized by honey-combed open or red clay-filled voids in the rock mass with horizontal and vertical drainage channels, sink holes and caves. A major normal fault zone passes beneath the western abutment of the dam with significant vertical displacement.

5.5.2 Sewer Line Installation

120 feet of 8-inch-diameter SDR 26 PVC sewer pipe and 120 feet of 8-inch-diameter SDR 35 PVC sewer pipe were laid in a blasted trench approximately five feet deep, following City of San Antonio specifications (Reference 4, Figure 5.8). The trench was difficult to excavate because of the irregularity of the rock quality horizontally and vertically. On occasion, a standard blast load would fail to break the hard caprock, but the following blast with a slightly heavier charge would blow out because of soft ground conditions. The excavated trench was lined with gravel, and pipe sections were connected by belled joints with stiff rubber gaskets. Three precast concrete manholes were spaced at 100 foot intervals along the pipe line. The precast manhole sections were joined to one another with rubber gaskets on horizontal mating surfaces, and cement grout on interior surfaces. The manhole-to-pipe joints were rubber boots clamped to the outside of the pipe, with cement grout covering the joint between the end of the pipe and the concrete inside the manhole. The pipe sections were mandrel and air-tested after installation, and the manholes were vacuum tested with the vacuum seal fitted below the

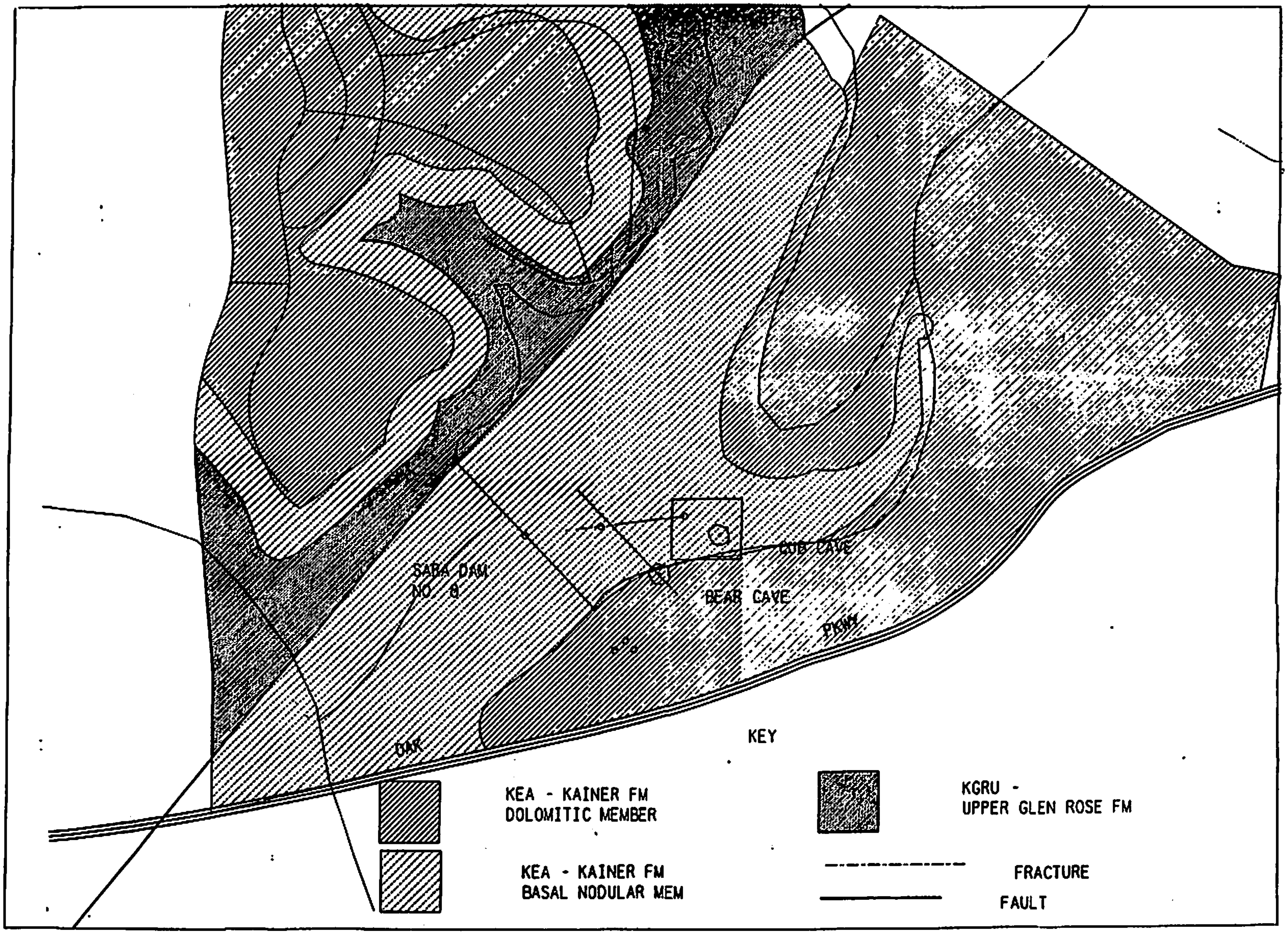


Figure 5.7 - Stone Oak Site Geologic Map

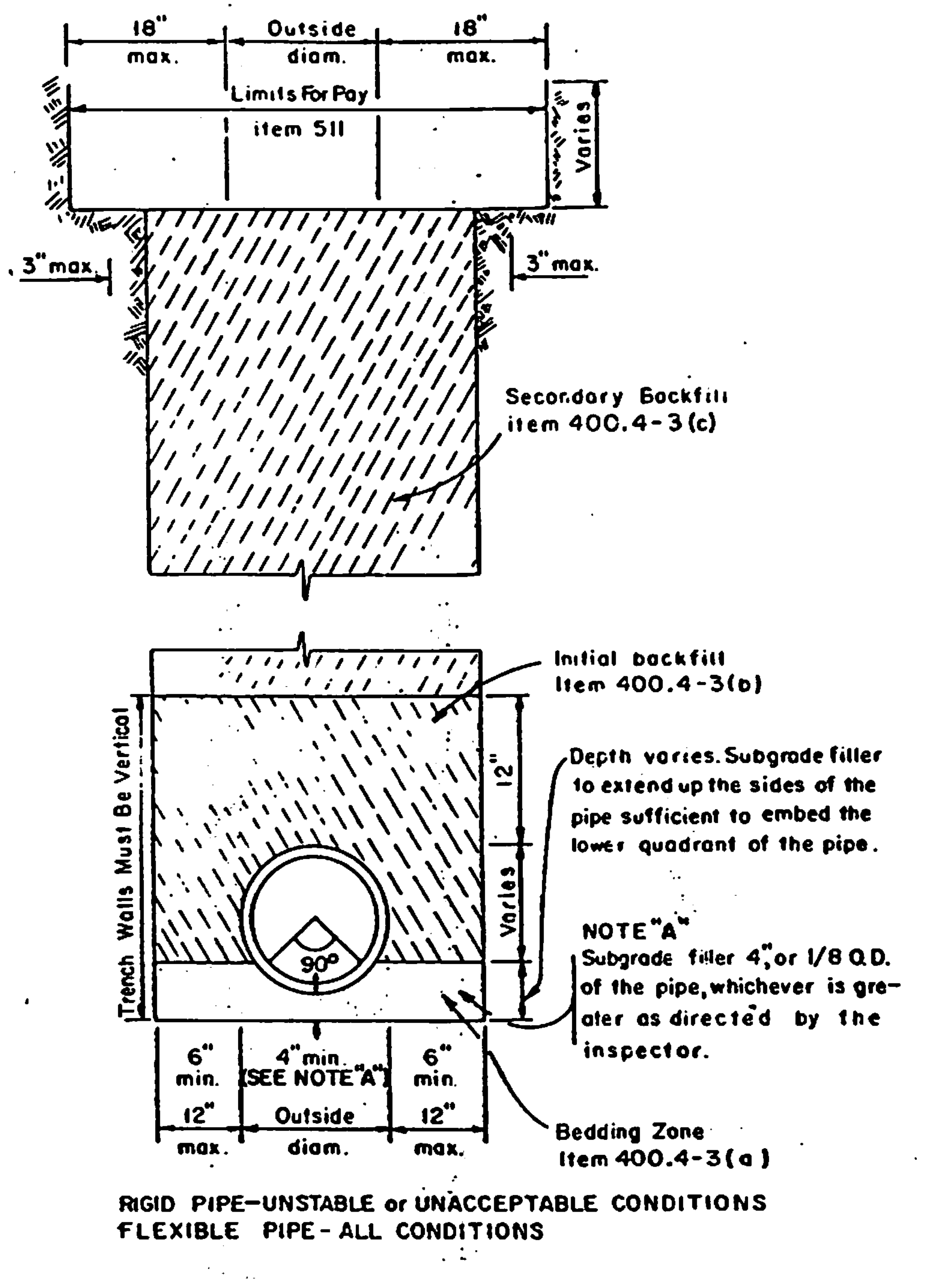


Figure 5.8a - City of San Antonio Pipe Installation Specifications

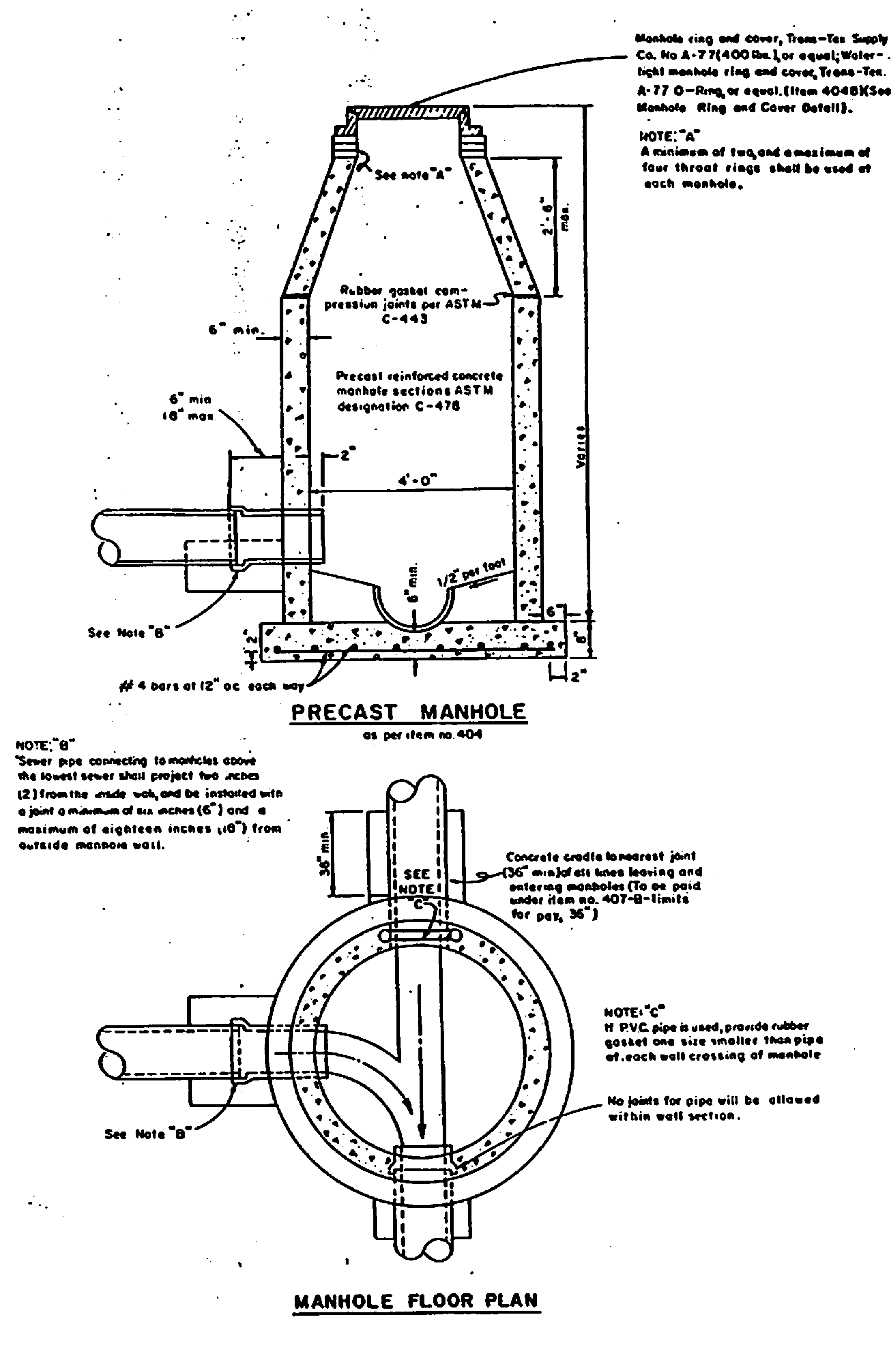


Figure 5.8b - City of San Antonio Manhole Installation Specifications

"doughnut" concrete extensions. All pipe and manhole seals were found to hold air pressure (+3.5 psi and -3.5 psi gage respectively) satisfactorally.

Elevation control for the manholes and pipe sections was maintained throughout the project. Threaded steel rods were connected to the top of the pipe at stations 1 + 50 and 2 + 50, and isolated from the surrounding fill with 2-inch-diameter PVC standpipes. The elevation of the manholes and tell-tales was measured at the time of air testing and checked periodically thoughout the project. Elevations were referenced to a benchmark in bedrock at station -1 + 00. No significant elevation change from blasting was noted.

5.5.3 Sewer Line Testing

Blast arrays for vibration testing of the sewer line were detonated along four major axes (Figure 5.9). Lines A, B and C ran perpendicular to the pipe sections at stations 2 + 50, 2 + 00 and 1 + 50 respectively. Line D ran parallel to the pipes from station 0 + 80 towards the northeast. All blast holes were two inches in diameter and were drilled by track-mounted rigs with impact drills powered by compressed air. Production blasting holes were drilled to a depth of 10 feet, then loaded with one cap inserted in 1/2 stick of dynamite (approximately 1/4 lb. of explosive), and approximately 10 lbs. of ammonium nitrate-fuel oil (ANFO) blasting agent (eg. Austinite 15 ANFO, Austin Powder Company). Each hole was stemmed (backfilled) with 2 to 4 feet of gravel or soil compacted in place. Test holes associated with the production blasting were also drilled to a depth of 10 feet, then loaded with charges ranging from 1/2 lb. of dynamite to a regular blast load of 10 lbs. of ANFO. Delays were nominally 25 milliseconds each.

The intent of the blasting program was to reproduce as closely as possible the vibrations generated by production blasting for sewer lines and highway construction. The blasting on each line began at a distance of at

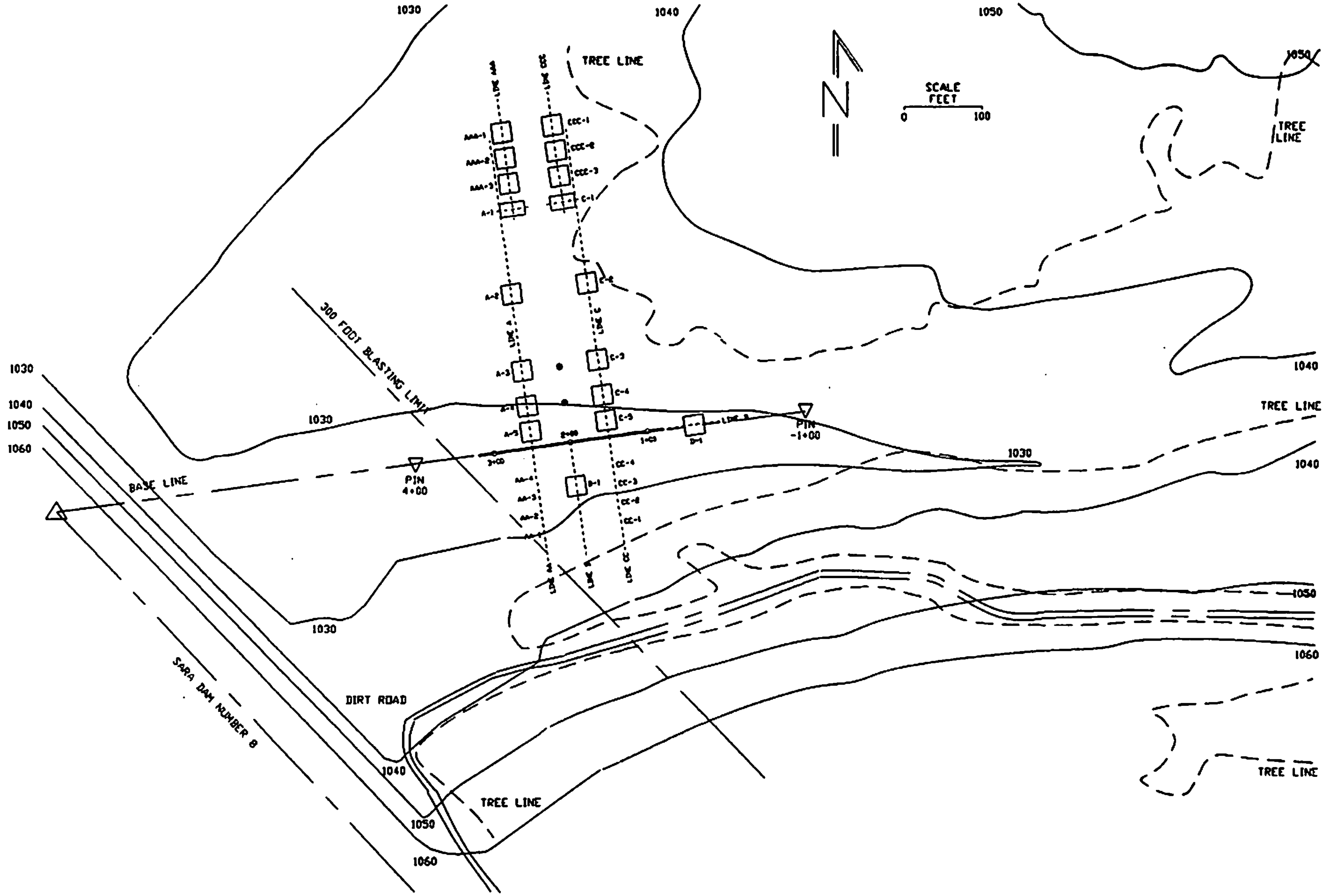


Figure 5.9 - Test Pipeline Plan and Blast Arrays

least 100 feet from the pipes and moved towards them. This blasting sequence would produce the most severe loading to the pipes because vibrations would be transmitted through intact rock. Blast array delay patterns and spacings between blast holes were chosen to reproduce actual production blasting operations.

The ground conditions, hole depth, charge weight, stemming and delay were documented for each blast hole. Colored slides were taken of the blasts with charge weight exceeding 2 lbs., and of the ground displacement resulting from each major blast. Color video was taken of several of the blast sequences.

Testing began with the detonation of three linear arrays (AA-1 through AA-3, Figure 5.9). Each array included nominally 8 blast holes spaced at 3 feet apart. Test holes were drilled and loaded with charges ranging from 0.2 lbs. to 0.8 lbs. adjacent to the center of each array. The blasting sequence began with the test holes, in order of increasing charge weight. The production blasting began with the two holes most distant from the pipe line, and continued toward the pipes with groups of 2 holes at 25 millisecond delays. The final array closest to the pipes was not detonated until line A shooting was finished, to preserve the integrity of the rock surrounding the pipes during blasting along line A. This initial, relatively low-energy sequence of blasts provided an opportunity to test the operation of the equipment and refine blasting procedures.

Ground instrumentation for recording the blast vibrations included three-dimensional groups of individual vertical and horizontal 8-Hz geophones spiked into the soil for the surface arrays, and special water-proof three-dimensional geophone packages for vibration measurements in the pipes and in boreholes (Appendix B). The packages were held in place with a special pneumatic system developed for the project.

The next blast array to be layed out (A-1) was the 12 foot by 32 foot by 17 foot deep excavation for the fiberglass test tank located on line A on the opposite side of the pipes from the line AA blasting. The center of the array was 306 feet from the pipe line. The work at this location served double duty by fracturing the rock for the tank excavation and providing vibrational loading for pipe response testing. The rock near the surface was relatively hard limestone, with a clay layer beginning at 7 feet. Holes were drilled on a 5 foot by 6 foot grid, with 5 test holes at either end of the array. The center of the array was displaced 16 feet towards the northeast from line AA to avoid a depression in the bedrock nearby which would cause irregularities in the wave transmission.

The excavation was blasted in two sections, beginning on the northeast end. The holes in the first blast section were loaded between 17 feet and 10 feet below the ground surface with 10 feet of stemming. These holes blew out into the soft clay layer at about 10 feet depth and failed to throw up the hard cap rock. The holes in the second half of the array were loaded to within 4 feet of the surface, and broke the surface rock nicely. Four holes were drilled in the first half of the array and detonated at a later date to finish the fracturing of the rock for the tank excavation.

The center of the next array (A-2) was located 200 feet from the sewer line (Figure 5.9). The surface rock was relatively-hard limestone with some weathered zones. The array was 25 feet square with 10-foot deep holes drilled on a 4 X 4 grid (8 foot spacing). Six test holes were drilled in two rows parallel to the production holes on the south-west side of the array. One hole was loaded with 1 stick (1/2 lb.) of dynamite, another with 4 sticks (2 lbs.) and the remaining 4 holes with a total of 44 lbs. Three-dimensional geophone arrays were placed on the ground surface adjacent to the array, between the array and the sewer line and adjacent to the sewer line at station 2+50. Instrument packages were also placed in the manholes, in the pipe and in borehole number 10 at 10 feet and 20 feet below the ground surface.

The first records taken for this array were hammer hits adjacent to the array and next to the geophone array closest to the blast. The 1/2 lb. charge was then detonated, followed by the 2 lb. charge. Finally, the 4 holes with production loads (44 lbs. total) were detonated together with no delays. Two more hammer hits adjacent to the blast array were recorded to complete the initial testing for this array.

Production blasting at array A-2 was scheduled for Monday, April 8, however storms on April 5 and 6 dropped approximately 10 inches of rain on the watershead of Mud Creek, totally innundating the site. The blaster's air drill rig, parked on the dirt road to the northeast of the array, was covered with 10 feet of water. After the water receded (approximately 5 days), the compressor was removed from the site and the engine rebuilt. The instrument packages in the pipes and borehole were also covered with water during the flood, but the sensors were waterproof and did not suffer damage. Water had seeped into the pipe system through the manhole cover gasket, and the pipes were pumped dry to obtain access to the instruments. All packages were removed and cleaned before reinstallation at their respective locations.

The two packages in borehole 10 were removed for cleaning on April 9, while the water surface in the creek bed was still above the casing stub sticking one foot above the ground surface at the hole. The inside of the casing was dry, but water was flowing laterally in the void near the bottom of the hole. The two flow regimes at this location (ponded water on the surface with slow infiltration and subsurface stream flow) are typical of drainage areas in the lower Edwards group, underscoring the importance of preventing leakage from tanks and pipes.

Effects of the flooding were also observed in the area blasted for installation of the pipeline. The backfill around manhole number 3 (station 3+00) had settled approximately 6 inches. At the edge of the excavation, open cracks and fissures in the rock had been washed clean by water flowing into the ground.

After a 12-day lapse in testing for equipment repair and site cleanup, blasting resumed on April 16. Shot holes for array A-2, which had been drilled before the flood, were blown dry and loaded. Six additional test holes were drilled on the northeast side of the array. Vibrations from hammer hits between the blast array and the first instrument package were recorded. Test blasts of 1/2 lb., 2 lb. and 40 lb. were detonated. The second set of tests blasts allowed for comparison of the ground and pipe response after the flood with the pre-flood response. Production loads were 10 lbs. of ANFO per hole detonated by 1/2 stick of dynamite with 5 feet of stemming. Delays (25 milliseconds apart) are shown in Figure 5.10.

The results of the array A-2 testing were positive, and lead to the development of a standard procedure for test blasts that was followed for the rest of the project. As expected, ground vibration amplitude at a given location generally increased with the increase in charge weight, and the ratio of response amplitude between one location and another remained relatively constant for all blasts in an array sequence. The truth of these general amplitude relationships was demonstrated by the success of the recording system operators in choosing the proper amplification settings on the fixed-gained preamplifiers for each data channel. If these principles had not been operational, and if the operators had not been able to incorporate them into their (impirical) choice of gain settings, a large

Production hole (with delay sequence) –

Test hole –

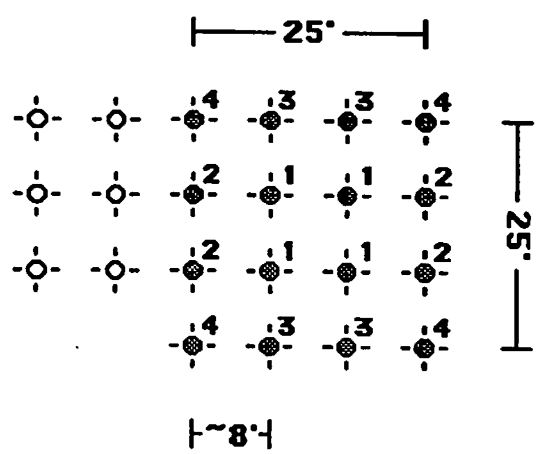


Figure 5.10 - Test Blast Array A-2

portion of the data would have been of poor quality due to low signal-to-noise ratio (low gain setting) or signal clipping (high gain setting). The extensive experience gained in predicting the response level of 32 components of ground motion to 7 levels of excitation gave the operators significant insight into the behavior of the ground system under dynamic loads.

Array A-3 was centered 100 feet from the pipe at the top of the slope leading down to the creek bed, and approximately 4.5 feet above the creek bed. The near-surface rock was layered alternating hard and soft limestone. Six test holes were drilled on the northeast side of the array, and loaded with three 1/2 lb. test charges, one 2 lb. charge. and 40 lbs. in four holes. Hammer hits between the array and the first instrument location were followed by detonation of the test shots and the 160 lb. production shot. The ground response to the first 1/2 lb. test blast was similar to the hammer hits. The response to the second and third 1/2 lb. test blasts was, however, more similar to the response of the subsequent larger shots. Evidently, the first test blast shifted the ground materials enough to change the vibrational characteristics of the material surrounding the blast array.

The next array (A-4) was located 50 feet from the pipe line. Six test holes were drilled parallel to and to the northeast of the array. The ground surface rock was relatively hard limestone with interbedded weathered zones. One of the holes caved in after drilling, and was only partially loaded, while another hole could not be finished because of soft, caving rock. The blast array was drilled on the slope leading down to the creek bed. The test holes were displaced by one blast hole row to the north to avoid boulders at the edge of the creek bed. A 6-inch-deep pool of standing water remained from the flooding between the blast array and the pipe line.. Test excitations included hammer hits, three 1/2 lb. test blasts, a 2 lb test blast and a 40 lb blast in four holes with no delays.

The next charge to be detonated was a four-hole reshoot of the first 1/2 of array A-1. The loads for the original blast were insufficient, and the blast had failed to break the surface. The four holes were drilled to a depth of 8 feet and loaded with 6 lbs. of blasting agent each. The blast fractured the surface rock, but did not blow out.

The final large blast series on line A (Array A-5) was centered at 22 feet from the pipe. The closest row of holes was 10 feet from the pipe line. The ground conditions were alternating hard and soft rock. Response was first recorded to hammer hits at three locations within the array. Test shots and production blast loads were similar to previous arrays.

Damage to the pipe system from this blast array was minimal, even though the rock on the side of the pipe trench excavation was displaced toward the pipe. Minor cracking of the mortar seal between the pipe and the manhole at the west end of the manhole at station 2 + 00 indicated some relative motion between the pipe and the manhole, however an air test of the pipe and manhole and pipe indicated that the rubber gaskets were still intact. The pipe-to-manhole connection at station 3 + 00 was also cracked, but less than the connection at station 2 + 00. Although displaced slightly from its original alignment, the sewer system would have continued to function properly.

The final array on line AA (AA-4) was laid out with eight holes at 3-foot spacings, 10 feet deep. Soft ground was encountered in some of the holes from 6 to 7 feet below the ground surface. The last hole was 10 feet from the sewer line. Additional test blasts were included in this array to determine the effects of varying the test blast charge weight. 5 test holes were drilled. The first three holes were loaded with 1/2 lb., 1 lb., 1 1/2 lb., respectively, then two holes were loaded with 1 lb. each detonated together for a total of 2 lbs. Hammer hits were recorded near the center of the array. The production array holes were detonated in pairs from south to north (four 25-millisecond delays). Some additional cracking of pipe-to-manhole mortar was noted in manhole number 2 as a result of this blast. The pipe section from 2+00 to 3+00 passed the mandrel test after array A-4 was detonated.

The blast testing on line A was successful, and good quality data was obtained. In order to assess the effects of ground conditions on response of the system, arrays were laid out along line C, parallel to line A, at the same distances from the pipe line. Test blasts were repeated and array sizes and productions loads were identical to those on line A. Square production arrays were shot in a cross delay pattern, opening with the four center holes and ending with the four corner holes, as on line A.

The blasting on line C began with the tank excavation, 32 feet by 12 feet in plan. The southeast corner of the array was 300 feet from the pipe on line C. The top 10 feet of the ground profile was hard, grey limestone, with a reddish clayey layer of weathered rock below. One line of three test holes was drilled 40 feet to the east of the array, and another line was drilled on the west side adjacent to the array. The production holes were drilled to a depth of 17 feet in three rows and five columns at 6 foot spacing. Each hole was loaded with 22 lbs. of ANFO.

Response to hammer hits at the center edge of the array was first recorded. Test shots with 1/2 lb., 1 lb., and 1 1/2 lb. of explosives were detonated on the west side of the array. The three test holes to the east of the array were loaded with 22 lbs. of ANFO each, and detonated with a delay of 25 milliseconds between each hole to simulate the time and space distribution of energy for the production blast. In the production blast, the middle column was detonated first followed by the two adjacent columns and the outer two columns. Good rock fracture was obtained with little fly rock.

A standardized technique for test shots was designed and implemented for the remaining arrays associated with the pipeline. Array C-2 was located 200 feet from the pipeline. Eight test holes were drilled on the west side of the array and nine test holes were drilled on the east side. Test holes were loaded with 1/2 lb., 1 lb., 1 1/2 lb., 2 lb. and 40 lb. (four holes) of ANFO, respectively. The 2 lb. charge was loaded in one hole on the west side and in two separate holes on the east side. The test blasts on the west side were shot with no delays, while the holes on the east side were shot with 25 millisecond delays between 1/2 lb. (1 stick) charges, to determine the effects of the delays on the energy distribution in the ground motion with respect to time. The 40 lb. test blast opened at the corner hole and ended with the interior hole to move the blast away from the production array. In general, the time histories for the test blasts with delays were more complex, with a longer duration of higher frequency motion. The single test hole on the west side of the array with four sticks of dynamite blew out because of inadequate stemming.

Near-surface conditions at array C-2 were clayey weathered rock with soft ground at 7 feet below the surface for some holes. Three sets of three hammer hits were recorded; at the center of each test array and at the center of the production array. The test holes without delays were detonated first, followed by test holes with delays. Finally the production array was detonated with very little ground displacement. Array C-3 was centered at 100 feet from the pipe line. Test blast loads were identical to those for array C-2, and the testing sequence was also identical.

Heavy rains on May 3 and 4 again flooded the site. This time, however, the contractor removed his equipment from the site before the flooding. The flood water rose to about 4 feet above the casing stub on borehole 10 at its highest point, and the water level in manhole number 2 after the flood was approximately 2 feet. Instrument packages were again removed from the pipes and borehole, cleaned and reinstalled.

The second flooding provided interesting insights into the recharge mechanisms at the site. The pneumatic systems for locking the instrument packages in the borehole were stripped from them by the force of the infiltrating water. About 1/3 of the plastic tarp that had covered the borehole was sucked down into it. Evidence of recharge through the many blast-induced cracks in the surface rock was found to the north of the pipe between stations 2+00 and 3+00. Water did not pond in this area as it had in the previous flood.

The array layout and testing sequence for arr-y A-4 followed the procedures established for array A-2. Hard to medium hard limestone at the surface overlaid soft weathered rock layers below 5 feet at some locations. The 2 lb. test charge with no delays blew a 100 lb. boulder into the air. The 40 lb. test blast with no delays blew the pneumatic locking device off the top geophone package in borehole 10, probably because of gas pressures venting through a void in the rock.

Air pressure tests were conducted on the pipes and manholes at this point to assess the damage to the section between stations 2+00 and 3+00 from production blasting on line A, and to assure that line A production blasting had not altered the condition of the section between 1+00 and 2+00. All pipe sections passed the air pressure tests, (held 4 psi for 6 minutes) and the manhole sections passed the vacuum test. All manholes held a partial vacuum of -3.5 psi (gage pressure) for 1 minute. The pipe section from stations 1+00 to 2+00 also passed the mandrel test.

After pressure testing, blasting was moved to line CC. A linear array of eight holes (array CC-1) was drilled 100 feet from the pipe. The surface rock was medium to soft limestone with large voids in the southernmost two holes. Each of these holes took 22 lbs. of blasting agent. Five test holes were drilled on the east side of the array, and four test holes were drilled on the west side. The holes on the east side were loaded with 1/2 lb., 1 lb., 1 1/2 lb. and 2 lbs. (in two holes) with delays, and the holes on the west side were loaded with the same charges, without delays. Hammer hits were recorded at the center of the array, followed by test blasts and the production blast with the holes detonated in pairs from south to north.

A 25-foot-square array was next layed out on line D (along the pipe centerline) with its center at station 0+25, 75 feet east of station 1+00. The purpose of this array was to test the blast response of the creekbed and the response of the pipe to loads directed along its centerline. The hard rock surface layer was approximately 2 feet thick with soft layers and thin voids below. A large void at a depth of 6 feet in a hole on the west side of the array required seven shovels of gravel to stop the loss in blasting agent. Hammer hits were recorded at the center of the array. Test and production blast loads for this array were identical to those in array A-2.

The next array was CC-2, located 60 feet from the pipe. Hammer hits were recorded at the center of the array. Test and production blast loads and delays were identical to those for array CC-1.

A 25-foot-square array was laid out on line B, which intersected the pipe line at station 2+00. The array was centered at 57 feet to the south of the pipe. The ground surface rock was hard limestone, with 1- to 2-foot-thick voids at a depth of 6 feet in the holes on the north end of the array. These voids communicated with other holes in the array. Test and production blast loads and delays were similar to those in array A-2.

The final major array in the pipe test blasting was array C-5, centered at 24 feet from the pipe. The southernmost line of holes was 12 feet from the pipe. The surface rock was hard, with some voids in the holes at the northwest corner of the array. A void encountered in the hole at the southwest corner of the array took 30 lbs. of blasting agent to fill. Test and production loads and delays for this array were the same as for array A-2. The 40 lb. production blast with no delays lifted the surface rock well. The production blast threw a boulder approximately 2 feet in diameter 50 feet into the air. Blast vibrations moved manhole number 2 approximately 1/8 of an inch with respect to the surrounding soil. The pipe displaced towards the south with respect to the manhole, breaking the grout seal with an opening of approximately 1/8 inch. This separation reflected rotation and displacement of the pipe in a horizontal planewith respect to the manhole connection. Hairline cracks were observed in the mortar between the extension ring joints. No cracking was observed in manhole number 1. The backfill in the pipe trench was moved toward the south by the production blast.

Arrays CC-3 and CC-4 were shot next, with centers at 45 feet and 23 feet from the pipe, respectively. The southernmost two holes in array CC-3 had voids 1- to 2-feet-thick at a depth of 6 feet. Other holes had layered soft and hard rock. Array CC-3 had only four production holes, with one delay per hole, while array CC-4 loads and delays were the same as those for arrays CC-1 and CC-2. As with array AA-4, the blasts from array CC-4 did not break open the surface rock, but the blast did open a fissure perpendicular to the pipe. No additional damage to the pipe-manhole connection was observed after the final two blasts.

5.5.4 Underground Storage Tank Installation and Testing

The second half of the blast tests at the Stone Oak site focused on the response of steel and fiberglass underground storage tanks. Work began with blast array layout on lines A and C. 25-foot-square arrays were located and drilled with centers at 100 feet, 68 feet and 34 feet from the tank longitudinal centerlines. Only 8 test holes were drilled with each array, on the east side of the array. Loads were 1/2 lb, 1 lb., 4 lb. and 40 lb, with delays between each stick of dynamite for the small blasts, and between each hole for the large test blast. Hammer hits were recorded at the center of each production array. Production loads were 10 lb. of blasting agent per hole, with 5 feet of stemming, as before. All holes were drilled to a depth of 10 feet.

The hole for the fiberglass tank was excavated, along a centerline perpendicular to line A, to a depth of 12 feet. Approximately 1 1/2 feet of weathered-in-place rock covered hard grey fractured limestone which extended to a depth of 6 feet. The rock below was red-brown gravelly weathered-in-place limestone. The base of the excavation was grey medium-hard silty limestone. The near-surface limestone had been severely fractured by the blasting, and approximate 5 feet of the wall caved into the excavation the night after the hole was opened.

The tank for the first hole was a Xerxes 10,000 gallon double wall tank with manholes near either end (Appendix I). 1 1/2 feet of 3/8 inch diameter washed gravel was placed in the bottom of the hole and leveled for bedding. The clearance between the tank and the side of the excavation exceeded 2.5 feet, while the distance from the tank to the ends of the excavation was approximately 2.0 feet. The tank was removed from the delivery truck bed and lowered into the hole with a 22-ton hydraulic crane. A single lifting lug carried the entire tank weight. 3/8 inch gravel was rained along both sides of the tank until it was covered to the level of the manways, even with the surrounding ground surface. The gravel between the tank and the ground surface was approximately 2 feet thick. The center instrument package was strapped to the lifting lug, and it's cable was run to the manway on the east side. 3-foot sections of sonotube (cylindrical cardboard concrete forms) were placed over the two manways to maintain access to them, and the excavation was backfilled to grade. Approximately 200 tons of gravel were required. After placement of the tank, the tank level was checked by transit end-to-end and side-to-side and found to be true with in 1/4 inch in both directions.

The three waterproof instrument packages were installed on the floor of the tank; one at each end, and one in the middle. The packages were secured with adhesive on the bottom and weighted down with 25-lb. bags of lead shot on top (Appendix B). Instrument packages were also strapped to the bottom side of each manway cover, and the covers were bolted to the tank. The waterproof cables for the packages inside the tank were run out through a 4-inch-diameter port in the manway cover on the east end of the tank. Finally, the tank was filled to slightly above the 1/2 level (3.95 feet from the top) with potable water from the Hill Country Water Works hydrant near the site.

Individual surface instruments were spiked into the ground, as before, above both ends and the middle of the tank, and between the tank and the first blast array, AAA-1, which was centered at 100 feet from the tank. Test and production holes were loaded as described above. Near-surface ground conditions were erratic, with 2-to-3-foot-diameter boulders in a matrix of dark brown clay near the surface. Some of the boulders were thrown into the air by the production blast.

Ground conditions for the array AAA-2, centered at 68 feet from the tank, were similar to those for the previous array. During the 1/2 lb. test blast, the surface of the water in the tank was observed through a four-inch-diameter port in the manhole cover. No perceptible motion was observed on the water surface, however a faint echo of the blast compressional wave vibrations was heard in the tank. During the 2 lb. blast, no water surface motion was detected, but a perceptible "bong" was heard, like dropping a manway cover in place. For the 40 lb. blast, a very strong "bong", like hitting the tank with a sledge hammer, was heard. This blast barely broke the ground surface.

The near-surface rock for array AAA-3 was much more competent than the rock for the two previous blasts. The southern-most line of holes for this array was 21 feet from the tank centerline, or about 10 feet from the northern edge of the excavation. The 40-lb. test blast blew gas out through the blast holes on line C, indicating the presence of subsurface pasageways. The production blast blew out through the hole at the southwest corner of the array, and lifted the ground surface. The instrument package adjacent to the southernmost line of holes was thrown into the air, indicating vertical accelerations in excess of gravity. Pronounced surface cracks extended from the production blast to the northwest corner of the excavation.

An impulse response test was performed on the tank before removal. A 70-lb. rock was dropped from a height of 3 1/2 feet on the gravel covering the center of the tank. The audible ringing inside the tank from the impact continued for approximately 4 seconds.

Upon completion of the vibration tests, the water was pumped out of the tank and the gravel was excavated from around the sides of the tank. The instruments were removed, and the tank lifted onto the haul truck. The tank was washed down and checked visually for damage. A few small dents were found where the tank had been impacted by rocks during excavation and removal. No damage from blasting was found. The tank was pressure tested at the fabricator's yard, and no leaks were detected.

The excavation for the double-walled steel tank was similar to the first tank excavation. The weathered layered limestone extended to a depth of approximately 4 feet, with hard grey fractured limestone to 9 feet. The remaining 4 feet of material was soft red-brown clayey weathered rock with some gravel inclusions. A void zone approximately 1 foot thick was encountered between the bottom of the hard rock and the top of the soft weathered rock at the northeast corner of the excavation.

The bottom of the excavation was lined with approximately 2 feet of gravel, and the tank was set in place with a 35-ton hydraulic crane. Gravel was rained around the ends and sides of the tank. The tank was checked by transit from end-to-end, and found to be 1/2 inch from level. Instrument packages were strapped to the lifting lugs on the top of the tank, and the excavation was filled with gravel to the ground surface level, with approximately 2 1/2 feet of gravel covering the tank. The manway on the west end of the tank was accessed through a 30-inch section of sonotube 4 feet long. The packages inside the tank at the east end and center were connected to the tank with adhesive and weighted with 25 lbs. of lead shot each. The package at the west end of the tank was strapped to the bracket for the access ladder. The instrument cables were run out through the manhole port to the surface. Finally, the tank was filled with potable water to within 44 inches of the top.

Ground conditions for array CCC-1, centered at 100 feet from the tank were similar to those at the array AAA-1. Also, surface rock at array CCC-2 was similar to the rock at AAA-2, and ground conditions at CCC-3 were similar to AAA-3. In general, the rock conditions for the two test series were similar, giving a good basis for comparison between the test results for the glass and steel tanks.

Ground surface instruments were installed between the array and the tank and above the two ends and center of the tank, and the test and production shots for array CCC-1 were detonated. Next, the arrays CCC-2 and CCC-3 were shot at 67 feet and 34.5 feet from the tank, respectively. Some dirt balls were blown out from the array CCC-2 production blast. Ringing in the tank was again audible during the blasts. The impact response of the tank was measured by dropping the 70-lb. rock on the gravel above the center of the tank after all arrays were detonanted. The elevation of the tank was measured after testing, and settlement was found to be neglegible. No significant damage to the tank was encountered.

6.0 DATA ANALYSIS

6.1 Preliminary Analysis

The individual time histories of digital data, recorded as described in section 5.1, were first converted from system units of amplitude to units of ground motion velocity (Appendix D). The converted data was then transformed to the frequency domain by standard signal conditioning techniques. The Fourier Transform plots generated by this technique indicate the distribution of velocity amplitude (ground motion) as a function of frequency, and are valuable in assessing the type of waves generating the recorded motion and the possible effects of the motion on the ground and structures.

The basic reduced data included the time history and the frequency response plots on a single computer screen or page (Figure 6.1, Appendix E). From this information, preliminary estimates of the data quality (signal-to-noise ratio), signal duration, amplitude distribution and frequency content were obtained to provide direction for further analysis.

The recorded data was generally of good quality. Some D.C. shifting was encountered in the data with low signal level and high analog amplification. This shifting was corrected in the reduction process. A few of the records showed clipping due to excessively high signal level and/or high amplifier setting, while several others had a low signal-to-noise level due to weak signal and/or low analog amplification. Some signal distortion in the frequency domain plots was generated by overdriving. One record was lost due to electronics malfunction. A total of over 330 events were recorded and reduced with 10,560 data traces from individual instruments (21,628,880 data points).

6.2 Secondary Analysis

Several higher-level displays were obtained from the preliminary data.

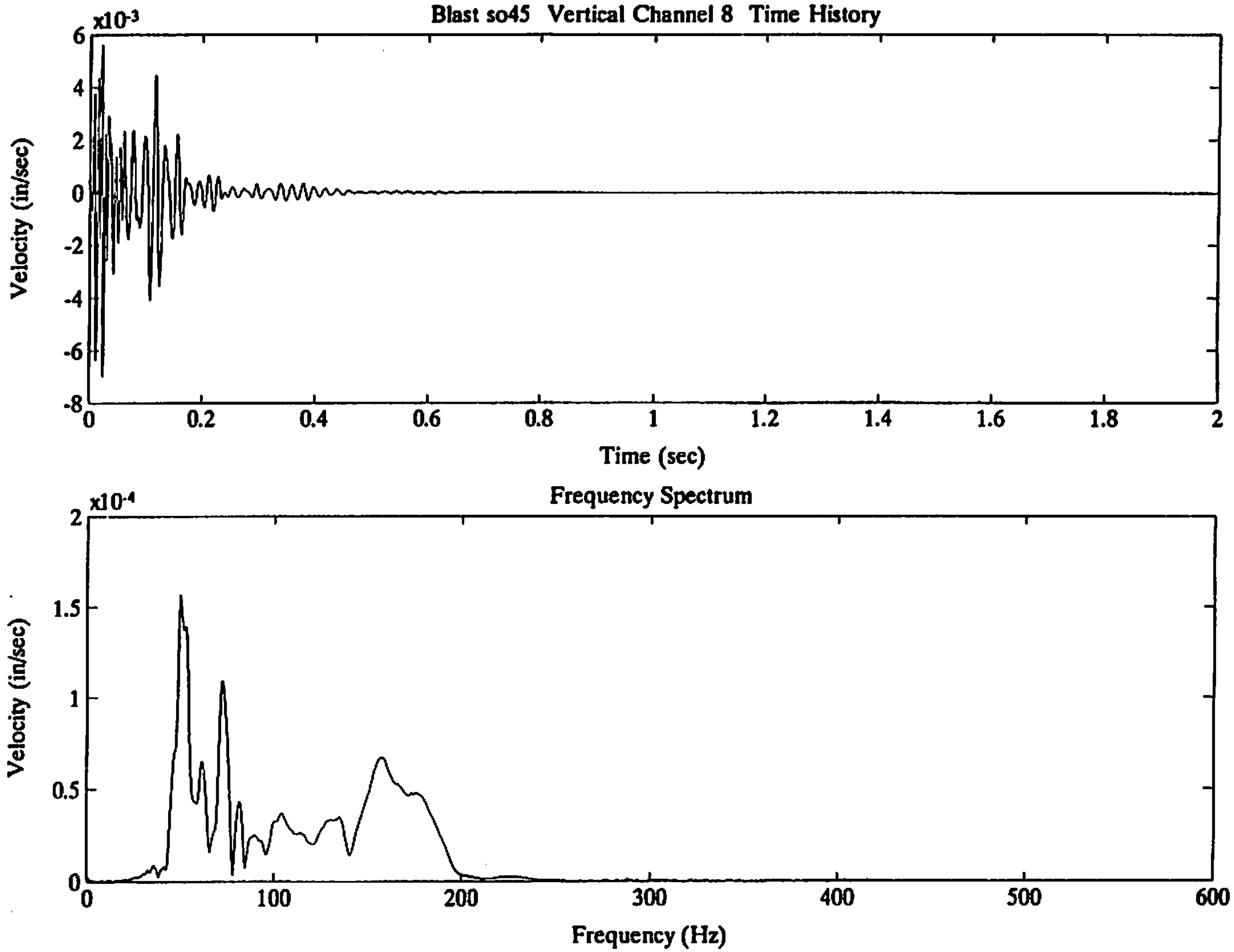


Figure 6.1 - Time History and Fourier Transform Display

Frequency domain response data for each record (blast event) were separated by direction of motion (vertical, longitudinal, transverse) and plotted in a three-dimensional displays of amplitude versus frequency and channel (Figure 6.2, Appendix E). Three plots were made per blast. Two-dimensional color contour plots of the same data were also generated (Figure 6.3). The two sets of plots complemented eachother, in that the three-dimensional plots demonstrated the continuity of energy from one trace (location) to the next at a given frequency, while the two dimensional plots allowed determination of exact frequency and amplitude at any point on the plot.

Additional functions were generated from the data after analysis of the plots described above. The application of these plots is explained in Section 7, and sample plots are displayed in Appendix E.

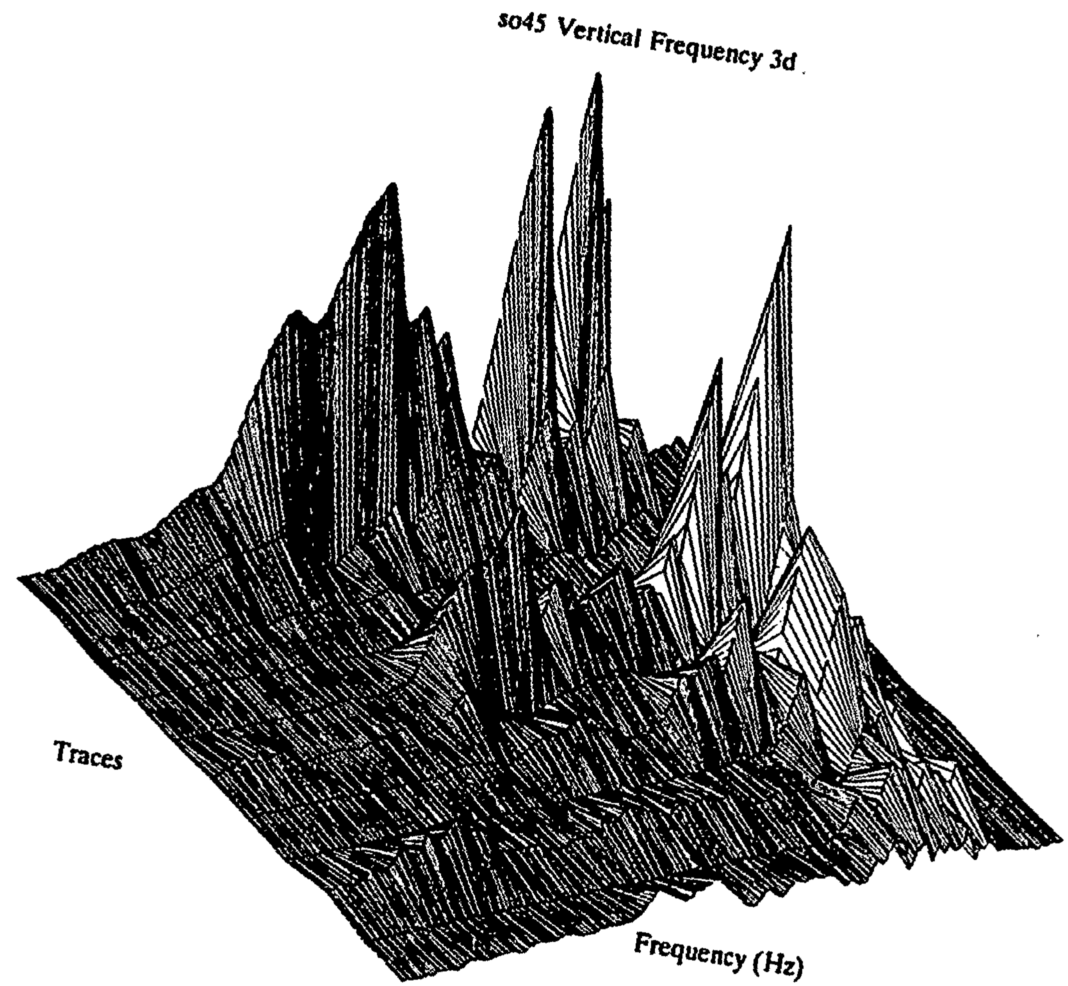


Figure 6.2 - Three-Dimensional Fourier Transform Display

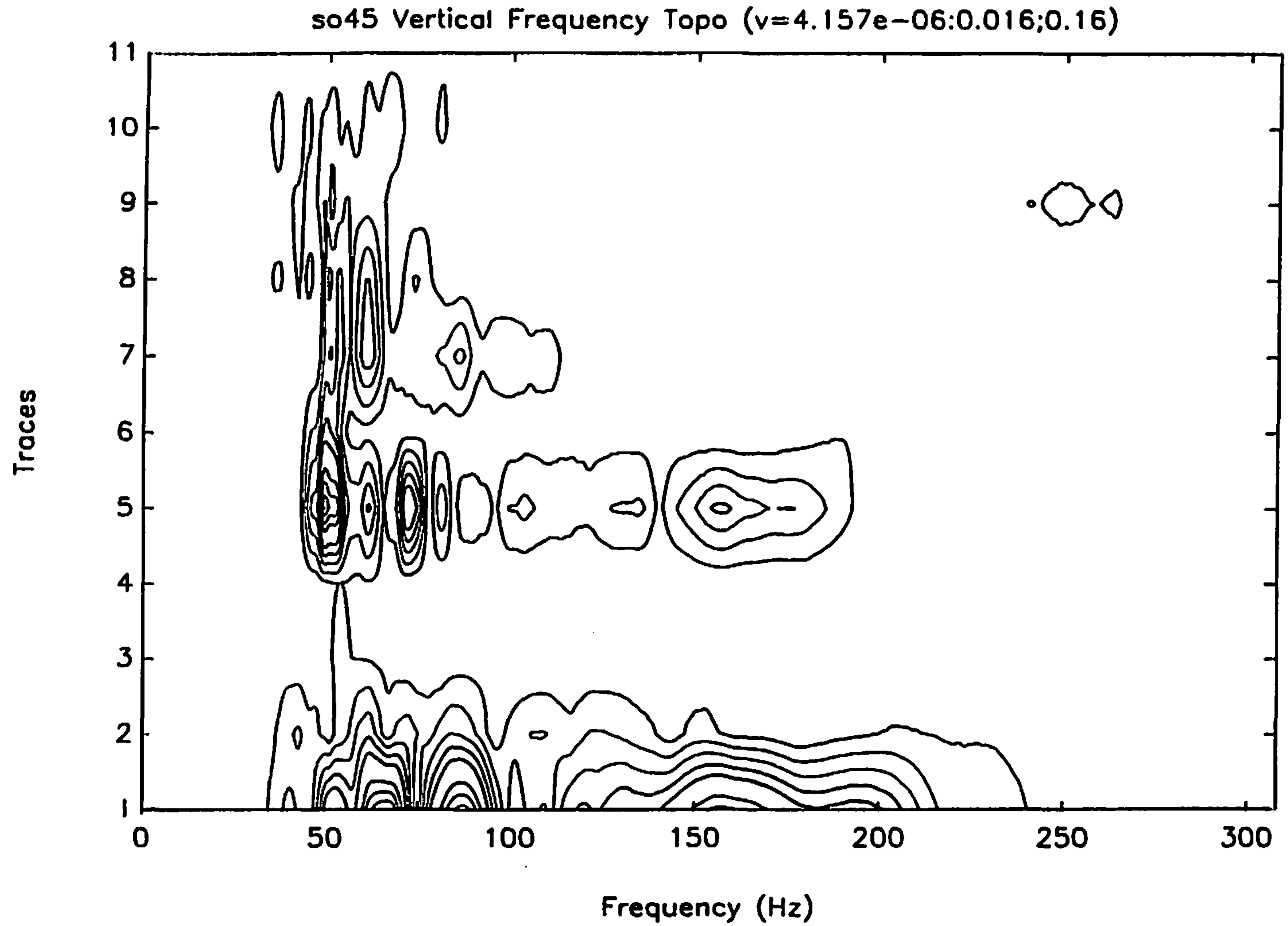


Figure 6.3 - Contour Fourier Transform Display

7.0 SYNTHESIS OF WORKING CRITERIA

7.1 Background

Predictions of ground vibration response amplitude can be developed by two techniques; (1) numerical models based on collections of measured blast response amplitude data (Equation 4.1, Reference 1), and (2) calibration of the blast response of the site by in-situ testing. The former method has been widely used to date because the numerical equations are accessible and easy to apply, while the technology required for site calibration studies has not been readily available. Unfortunately, the numerical models (1) are often derrived from a combination of data taken from many sites with widely varying energy sources, blasting techniques and geologic conditions, and the resulting relationships therefore contain a large amount of scatter.

Fortunately, recent advances in micro-electronics have made on-site measurement operations much more efficient, and results of data analysis are available immediately for planning of on-going operations. Improvements in blast monitoring and analysis techniques will hopefully increase the reliability of site response predictions improve the efficiency of blasting operations and decrease damage to engineered structures. Specific recommendations for implementation of the prediction technique are developed below.

7.2 Theoretical Basis

The experimental method for predicting site response developed in the present study is based on the principles of impulse testing and system identification which are employed in exploration geophysics, vibration analysis of mechanical systems and earthquake engineering (References 5 and 6). The physical system under study is subjected to a calibrated low-level impulsive excitation event (eg. hammer hit), and the time history of response is recorded at many points on the system. The ratio of the system response at each point to the excitation amplitude is then computed. This relationship (the frequency response function, FRF) completely characterizes the dynamic behavior of the system. It can be used to predict the response of the system to much larger events, as long as the higher level excitation does not significantly modify the response characteristics of the system (linearity requirement).

The site calibration technique employed in the present investigation follows, in general, the vibrational testing principles outlined above. The site response to low-energy excitation is measured first, then the response to production blasting is predicted from the measured response. The computational method for determining high level response from measured data in the present work is different, however, from that used in the classical procedure because the present system differs from the classical system in several important respects.

In classical testing, the amplitude of the load applied to the system is relatively small, and loads are non-distructive. The magnitude of the load can be measured accurately with conventional instrumentation. Also, the system response to excitation is approximately linear. The above two conditions are necessary prerequisites for determination of the classical FRF, which strictly defines the vibrational response of the system. By contrast, the high energy source in a blasting system cannot be conveniently calibrated in the field, because the energy-generating mechanism destroys the medium in which it is detonated and compromises any measuring instrument placed close to it. Also, the high concentration of energy at the blast location deforms and degrades the surrounding material, generating significant nonlinearities. As a result the classical FRF cannot be determined for the blasting system, and an alternate procedure must be developed.

One important system component which is often used to characterize the energy source and ground response to blast loading, is the weight of blasting agent. Charge weight is not, however, an entirely accurate indication of the magnitude of the energy source, because a given charge can generate various types and levels of ground excitation, depending on ground conditions, the amount and direction of ground throw, and the amount of blow-out. Characterization of the energy source by charge weight is further complicated by the fact that the energy generated by blasting is usually not truely impulsive. Charges in the various holes in the blast array are detonated at different times (delays) to lessen the peak vibration amplitude and move the blasted material in a pre-determined direction. Also, the blast excitation event is never repeatable, because any blast, no matter how small, changes the characteristics of the adjacent rock. Finally, the blast event is difficult to characterize because the ground vibrations are generated at the moving surface of a cavity which expands rapidly and then collapses or stabilizes. The exact nature of energy generated by a particular blast cannot, therefore, be predicted on the basis of charge weight alone. How, then, can we formulate a dependable indicator of the size of the energy release from a given blast? A more sophistocated measure of the relationship between the charge weight and system response, including the effects of the ground conditions between the blast and the measuring instrument, is required.

To fill the need described above, a new response prediction technique has been developed. It makes the link between measured low-level response and predicted response to production loading through the blast energy-dependent velocity response ratio (VRR). As a first step in the development of this function, the ground response to a low level excitation event (hammer hit and/or small blast) is measured simultaneously at many locations in the area of interest. The response to larger excitation events is then measured at the same instrument locations. The relationship between velocity amplitude and charge weight (VRR) is then computed for each instrument location and for the site in general. Once this relationship has been established, the response at specific site locations to future production blasting can be determined from the measured response to small blasts and the VRR as outlined in the following sections.

7.3 Criteria Development Procedures

7.3.1 Velocity Response Ratio

In the present study, the information necessary for development of site response prediction criteria was gathered at many locations with varying geology. The majority of the work was done at the Stone Oak site. Excitation levels for the vibration tests at Stone Oak ranged from a 20-pound hammer impact on a 6-inch-square aluminum plate to 1/2 lb. of dynamite in a single hole to 160 lbs. of blasting agent in a 16-hole array. From measurements of site response to various energy levels, relationships between the charge weight and the amplitude of response were developed for specific site locations and for the site in general (VRR plots, Figures 7.1 and 7.2). In these plots normalized velocity amplitude is a function of total charge weight. The relationship between normalized velocity and charge weight per delay is discussed in Appendix F.

Figures 7.1 and 7.2 show the ratio of the peak amplitude of response to a blast of a given charge weight to the peak amplitude of response to a 1/2 lb. test blast, as a function of total charge weight. The data in the sample

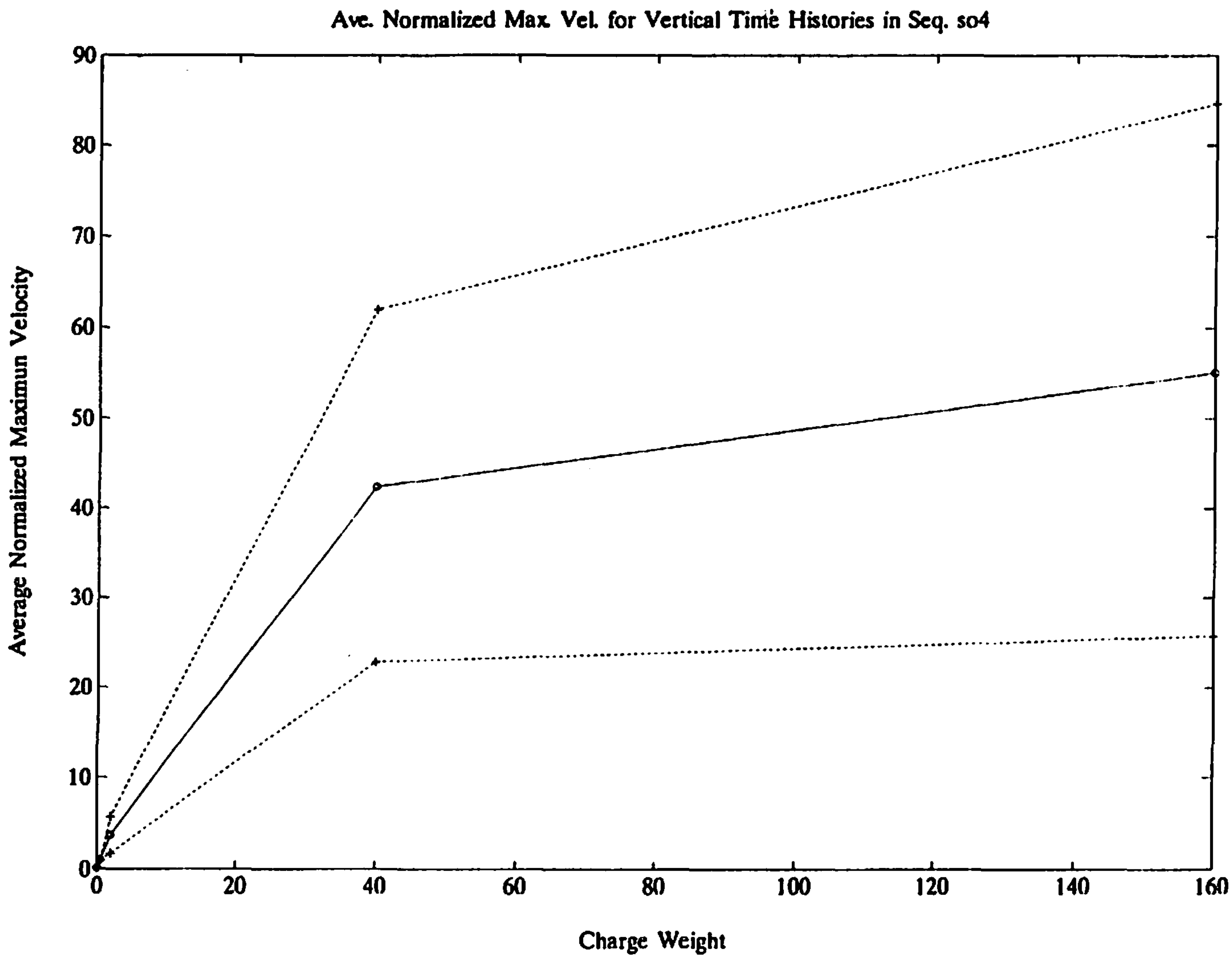


Figure 7.1 - Velocity Response Ratio Versus Charge Weight

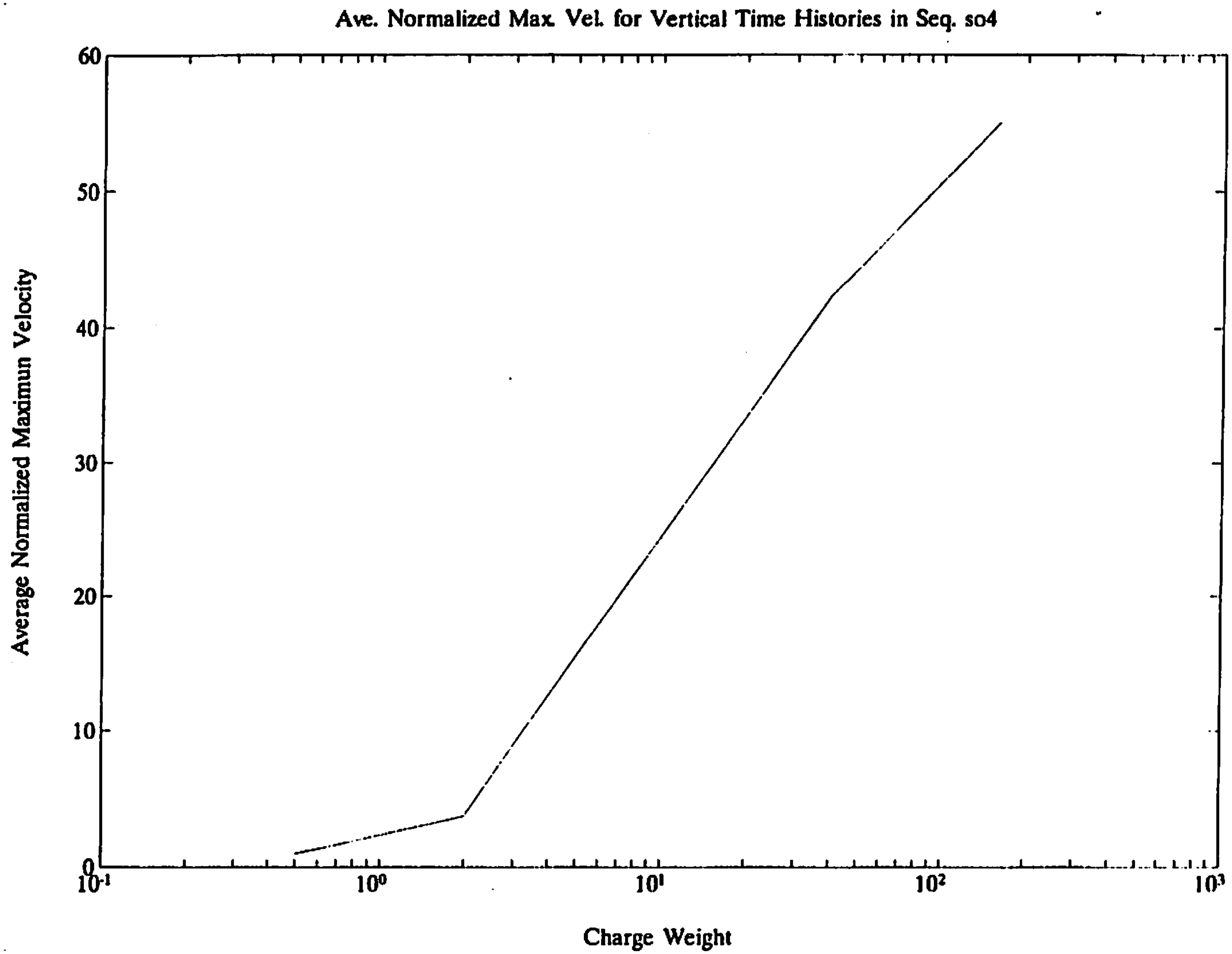


Figure 7.2 - Velocity Response Ratio Versus the Logarithm of the Charge weight

plots are the average of the responses of all of the 12 vertical instrument components measured for all the hammer hits and blasts at array A-3 (centered at 100 feet from the test pipeline). The peak amplitude was defined, for this application, as the average of the three highest peaks of the time history. The relationships are well-behaved, and lend themselves to blast response prediction applications. The data are plotted on arithmetic scales in Figure 7.1 to show the general characteristics of the relationships, including + 1 standard deviation and - 1 standard deviation. The data are plotted to a logarithmic horizontal scale in Figure 7.2 to demonstrate development of the underlying numerical characteristics of the data. In Figure 7.2, the relationship between the log of the charge weight and the normalized average velocity amplitude (VRR) is nearly linear above charge weights of 2 lb. The VRR for a charge weight above 2 lbs. would be computed as follows;

$$V/V_o(W) = 26 \log_{10}(W/1.5 \text{ lb.}) \qquad (7.1)$$

V = Velocity amplitude at the given charge weight

V_o = Velocity amplitude at a charge weight of 1/2 lb.

1.5 lb. is the x axis intercept of the straight line approximation to the data.

Relationships for other data (Appendix E) are similar.

To demonstrate the application of equation 7.1, let us predict the amplitude of ground response to production blasting measured by the vertical component of an instrument located 50 feet from the blast array A-3 (Station

(1) Determine the peak vertical ground response amplitude at station S-4 for a 1/2 lb. excitation event by averaging the response to three test blasts; $V_o = 0.085$ inches per second from the field test data.

(2) Compute the velocity response ratio for the production blast load of $W = 160$ lbs. blasting agent by equation 7.1; $VRR = V/V_o = 52.7$.

(3) Compute the predicted velocity response amplitude to the production blast; $V = V_o\ (VRR) = (0.085$ inches per second$)(52.7) = 4.48$ inches per second.

As demonstrated by the above example, useful relationships between charge weight and peak particle velocity can be developed from the field data, and the relationships can be used to predict response to blasts of various charge weight at a given site.

7.3.2 Normalized Velocity Response

Velocity response ratio plots such as the one shown in Figure 7.1 are appropriate for prediction of the ground surface response to blast vibrations on hard rock sites in the project target area (the ERZ). Relationships between the ground surface vibration amplitude and the amplitude below the surface were also obtained from the project data to predict the response of subsurface structures. Three classes of structures were considered; a) buried pipes, b) underground storage tanks and c) water wells. Figures 7.3, 7.4 and 7.5 show typical normalized relationships between peak velocity amplitudes at the surface and amplitudes at depth developed from the project data. Data for the buried pipes were obtained from the San Antonio River Authority Dam number 10 site and the Stone Oak Development site. Underground storage tank data were from the Stone Oak site, and water well data were

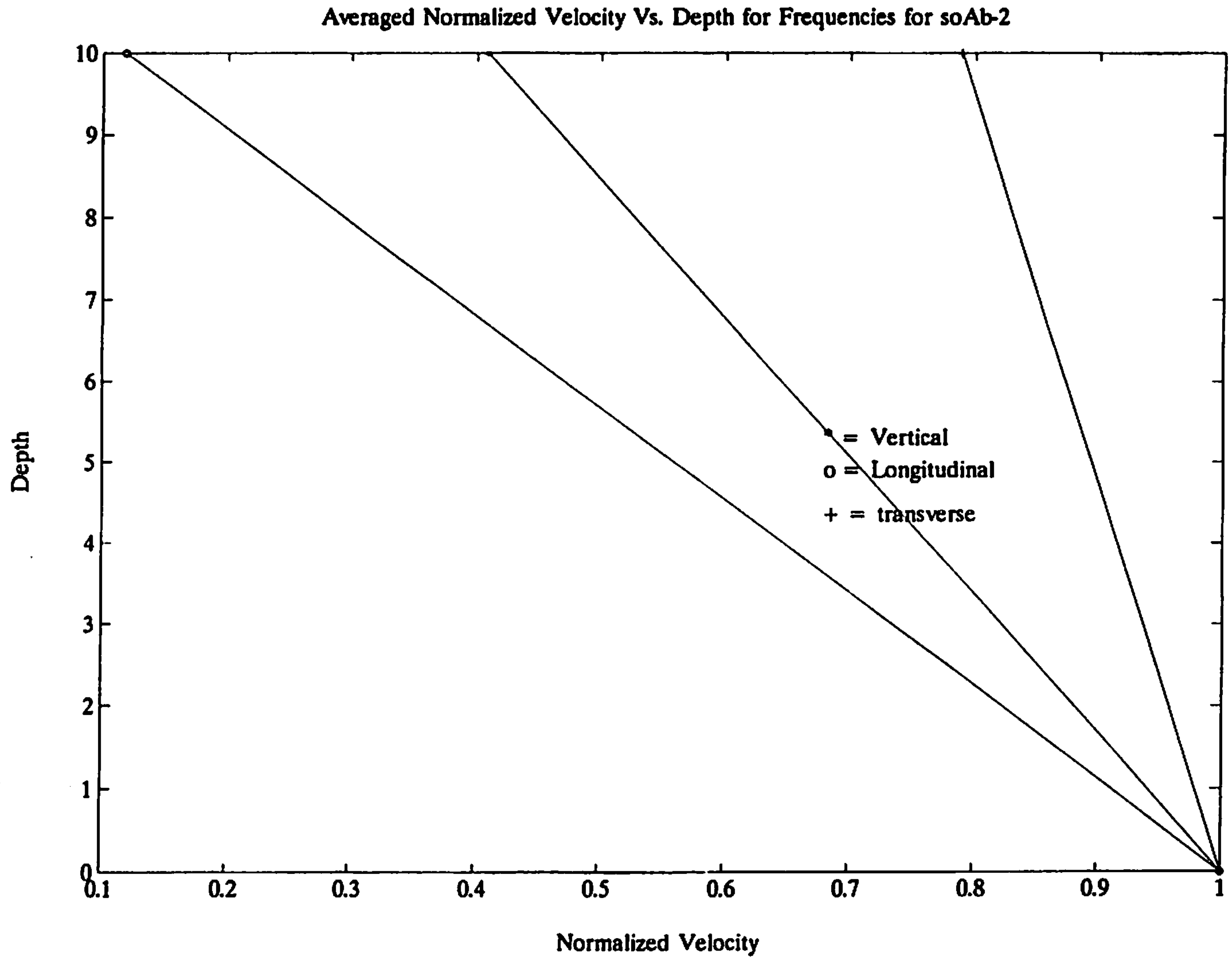


Figure 7.3 - Normalized Velocity Amplitude Versus Depth; Burried Pipe

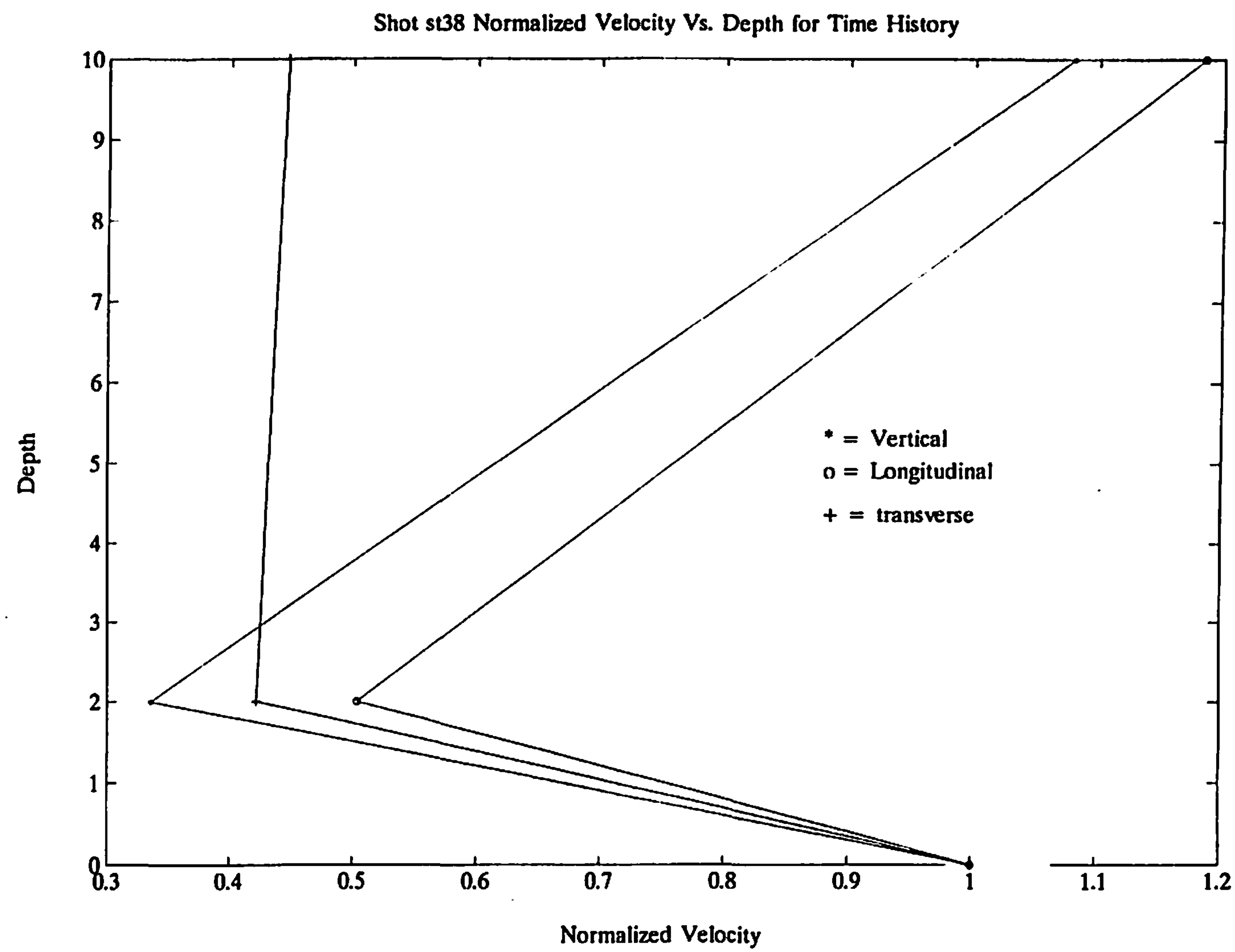


Figure 7.4 - Normalized Velocity Amplitude Versus Depth; Underground Storage Tank

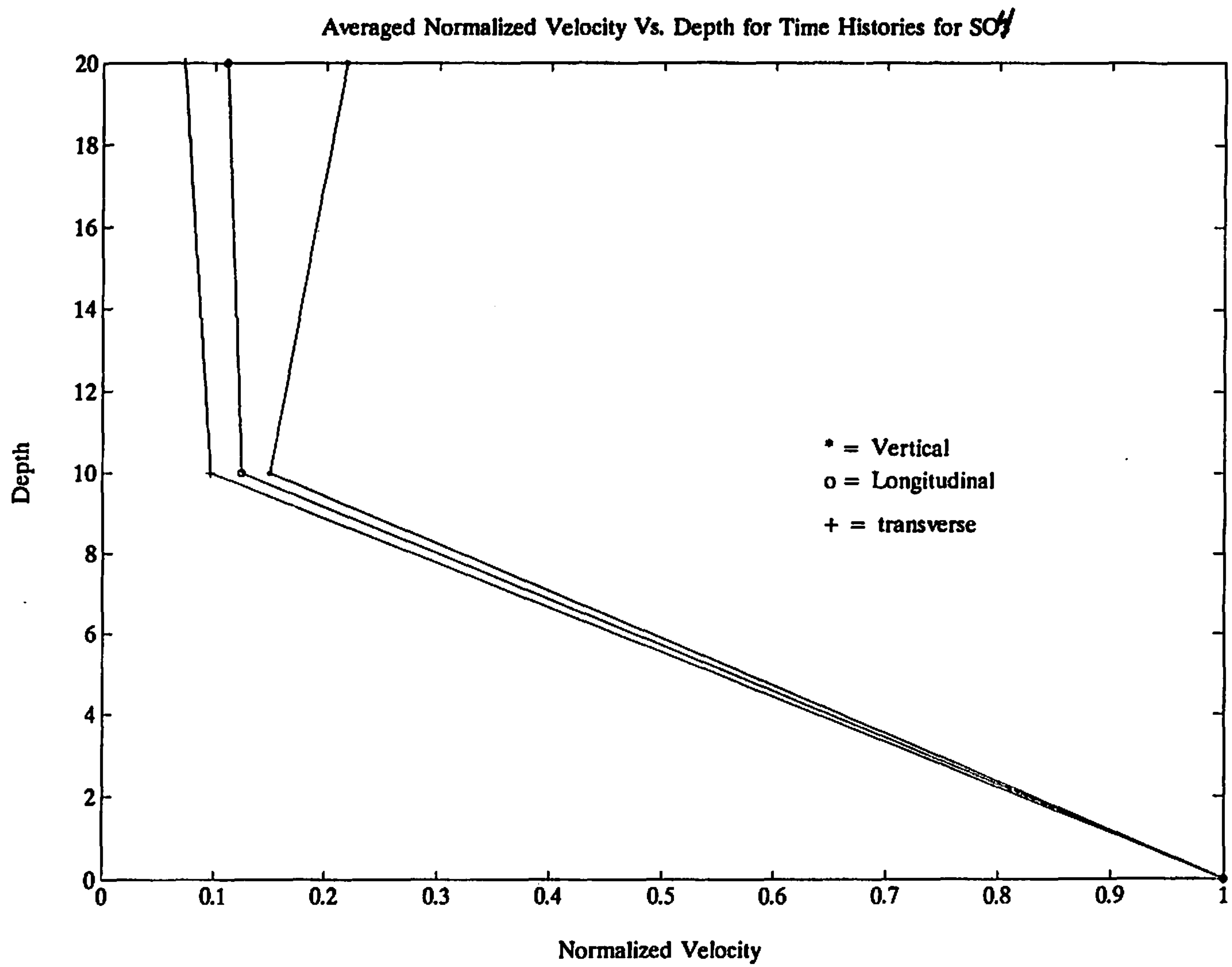


Figure 7.5 - Normalized Velocity Amplitude Versus Depth; Water Well

taken from the Judson/1604 site and the Stone Oak site.

7.4 Vibration Levels Associated with Structural Damage

Considerable time and effort have been devoted to determining the relationship between peak particle velocity measured at the ground surface and damage to structures. Most of the data has been developed from single location measurements of ground response to quarry blasting. The structures affected were, in most cases residential units of wood frame construction with drywall interior finish. The data indicate that in almost all cases, peak velocities in excess of 2 inches/second are required to cause noticeable damage, that is cracking of the drywall. Drywall cracking, in turn, reflects applied dynamic stresses exceeding the tensile capacity of light, unreinforced masonry.

A more sophisticated relationship has recently been developed which includes the simultaneous effects of ground vibration peak amplitude and frequency (Reference 1). According to this model, structures are capable of withstanding higher displacements at lower frequencies of ground motion than at higher frequencies, and higher velocity amplitudes at higher frequencies than at lower frequencies. This is a demonstration of the fact that for a given amplitude of harmonic motion, velocity level increases linearly with frequency. Since structural damage is most closely associated with velocity amplitude, threshhold displacement amplitude for damage should decrease with increasing frequency. The details of development of established damage criteria are discussed in Appendix G.

Although a great deal of experimental data has been gathered in relation to damage to above-ground structures, relatively little is available for buried structures. Enough similarities exist, however, to allow extension of the above observations to the case at hand. In many case, the subsurface structures most vulnerable to blast damage are those encorporating unreinforced concrete or masonry. In these cases, the peak velocity criteria of 2 inches/second in general and the frequency-related peak velocity may well be appropriate. Indeed, the experimental data obtained at the Stone Oak site indicate that damage to the mortar seals on the precast concete manhole extensions and the manhole-to-pipe seals was associated with peak particle velocities in the vecinity of 2 inches per second from blasts detonated within approximately 10 feet of the structures (appendix G). Blasts detonated at greater distances with lower peak particle velocities did not cause any noticeable damage. Although complete investigation of allowable peak velocity amplitude criteria for all types of buried structures was beyond the scope of the present study, results obtained indicate that 2 inches/second is probably a good maximum level for new structures which have been properly installed. Permissible velocity levels for older structures in questionable condition should be lower.

7.5 Summary of Procedure

The proposed buried structure response prediction method (Figures 7.6 and 7.7) combines the surface and subsurface normalized response relationships described above, and would be implemented as follows;

1) Obtain as-built drawings or descriptions of for target structures and locate them on the site. Determine the condition of the structures.

2) Place velocity sensors at key locations between the blast location(s) and the structures, and above buried structures. The number of sensors required would depend on the condition, complexity and importance of the target system.

3) Monitor response to one or more small blasts (eg. 1/2 lb. explosives)

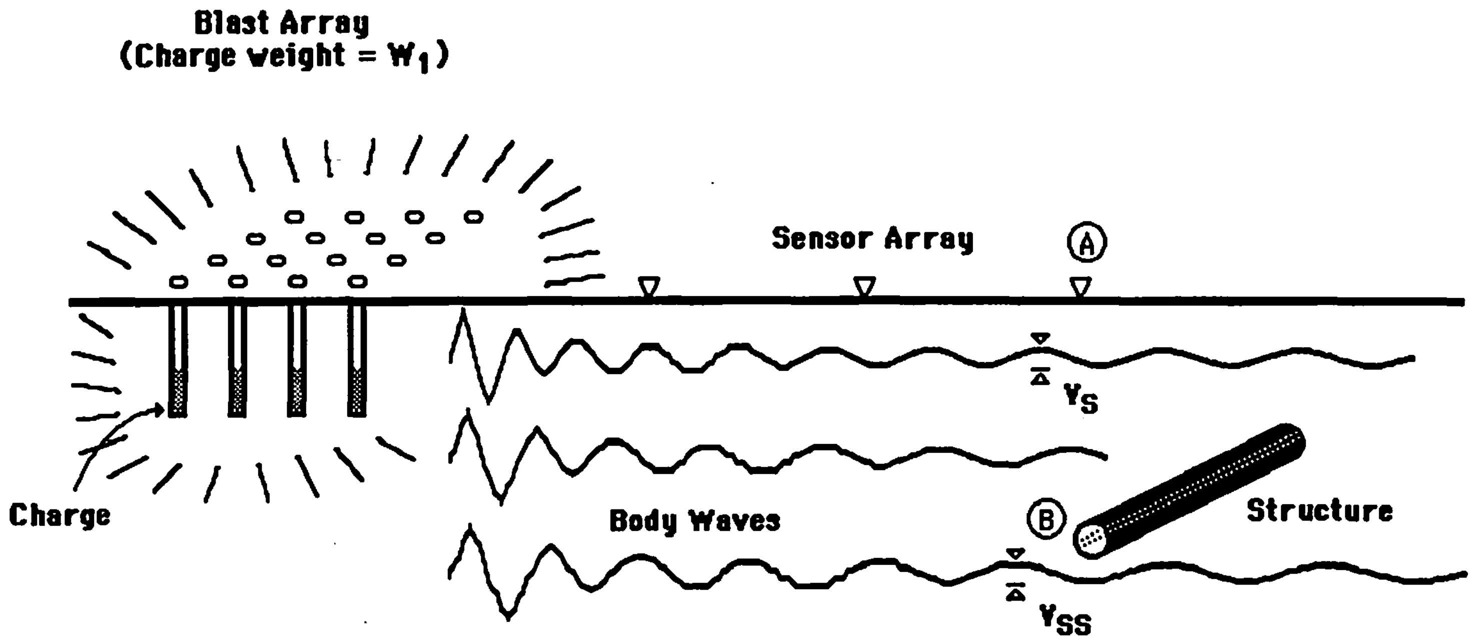
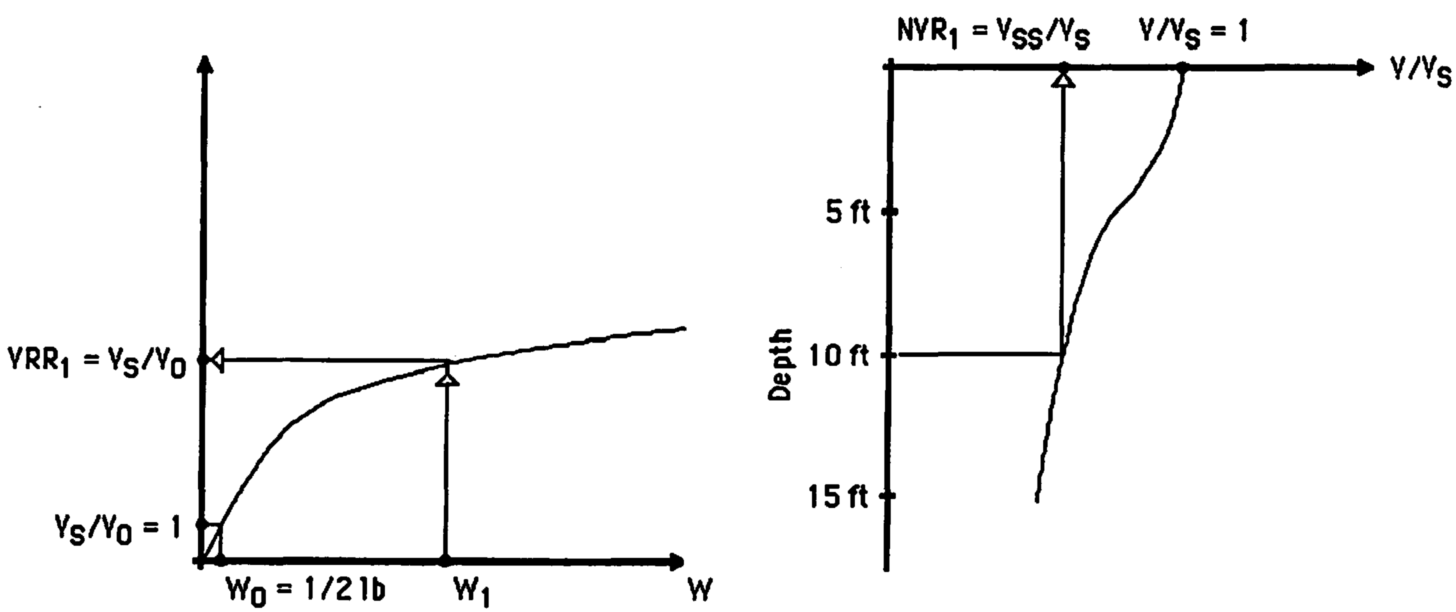


Figure 7.6 - Blast Vibration Transmission Through Ground



Velocity Response Ratio at (A)

Normalized Velocity Response at (B)

Figure 7.7 - Blast Response Prediction Functions

to determine the low-level response V_o at the instrument locations.

4) Determine the charge weight of the production blast, W_1.

5) Determine velocity response ratio for the production blast, $VRR_1 = V_s/V_o$, from the charge weight, W_1, and the VRR plot appropriate for local geology.

6) Compute expected peak surface velocity, V_s for production blast from the response ratio and the measured response to the 1/2 lb. blast, V_o.

$$V_s = VRR_1 (V_o) \tag{7.2}$$

7) Determine the normalized velocity ratio, $NVR_1 = V_{ss}/V_s$ at the depth of the buried structure, D_1, from a plot appropriate for the site geology.

8) Compute the predicted peak subsurface velocity, V_{ss} on the buried structure from peak surface velocity , V_s and NVR_1;

$$V_{ss} = NVR_1 (V_s) \tag{7.3}$$

9) Compare expected peak velocity on the structure, V_{ss} with manufacturer's damage criteria, and/or an assessment of the structural integrity of the system before the blast. Evaluate the safety of the proposed blast procedures. Modify the blast design or seek alternate excavation methods, if necessary.

8.0 Discussion of Results

The field studies and data analysis conducted by the investigators have demonstrated the feasibility of ground response predictions based on measured site response. The procedure can be used in situations where the proximity of the blast to the structure, or the sensitive nature of the structure and/or its contents warrant special attention. As the technique is implemented in a variety of geologic settings to predict the response of different structures, a catalog of data can be accumulated to improve our characterization of the interaction between the blast vibrations and the transmitting geologic media. As work progresses, the accuracy and precision of the technique will improve, until it is accepted as a standard tool for guaranteeing the quality of construction excavation work.

REFERENCES

1) Siskind, D.E., et al, "Structure Response to Damage Produced by Ground Vibration From Surface Mine Blasting," Report of Investigations 8507, Bureau of Mines, United States Department of the Interior.

2) Nicholls, H.R., et al, "Blasting Vibrations and Their Effects on Structures," Bulletin 656, Bureau of Mines, United States Department of the Interior, 1971.

3) Blaney, G., et al, "Geophysical Methods for Discrimination of Geologic Discontinuities in the Design of Hydraulic Structures," Second International Symposium on Design of Hydraulic Structures, Colorado State University, Fort Collins, CO, June, 1989 (in review).

4) City of San Antonio, Texas, Department of Public Works, Standard Specifications for Public Works Construction, January, 1981.

5) Baker, W.E., Westine, P.S. and Dodge, F.T., Similarity Methods in Engineering Dynamics, Hayden Book Co., Rochelle Park, N.J., 1973.

6) Luk, Y.W. and Mitchell, L.D., "System Identification via Modal Analysis," A.S.M.E. AMD, Vol. 59,1983, pp. 31-49.