43328 vol 3

INVESTIGATION OF DAMAGE TO STRUCTURES

IN THE M^cCUTCHANVILLE-DAYLIGHT

AREA OF SOUTHWESTERN INDIANA

Volume 3 of 3

Part VII: Experimental and Analytical Studies of the Vibration Response of Residential Structures Due to Surface Mine Blasting.

Part VIII: Dynamic Soil Property Testing and Analysis of Soil Properties, Daylight and McCutchanville, Indiana.

Part IX: Environmental Conditions Related to Geology, Soils, and Precipitation, McCutchanville and Daylight, Vanderburgh County, Indiana.

43328 Vol 2

OSM	

OFFICE OF SURFACE MINING

RECLAMATION AND ENFORCEMENT

TECHNICAL REPORT/1994

INVESTIGATION OF DAMAGE TO STRUCTURES

IN THE McCUTCHANVILLE-DAYLIGHT

AREA OF SOUTHWESTERN INDIANA

Volume 2 of 3

Part II: Geologic and Unconsolidated Materials in the McCutchanville-Daylight Area.

Part III: Blast Design Effects on Ground Vibrations in McCutchanville and Daylight, Indiana from Blasting at the AMAX, Ayrshire Mine.

Part IV: Vibration Environment and Damage Characterization for Houses in McCutchanville and Daylight, Indiana.

Part V: Racking Response of Large Structures from Airblast, A Case Study.

Part VI: Investigation of Building Damage in the McCutchanville-Daylight, Indiana Area.

US Department of Interior
Office of Surface Mining
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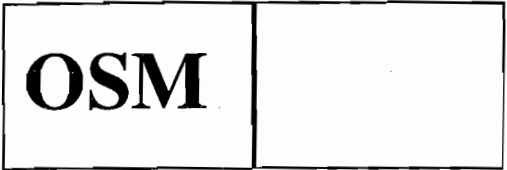Keltschl@osmre.gov

U.S. Department of the Interior

Part VIII

Dynamic Soil Property Testing and Analysis of Soil Properties, Daylight and McCutchanville, Indiana.
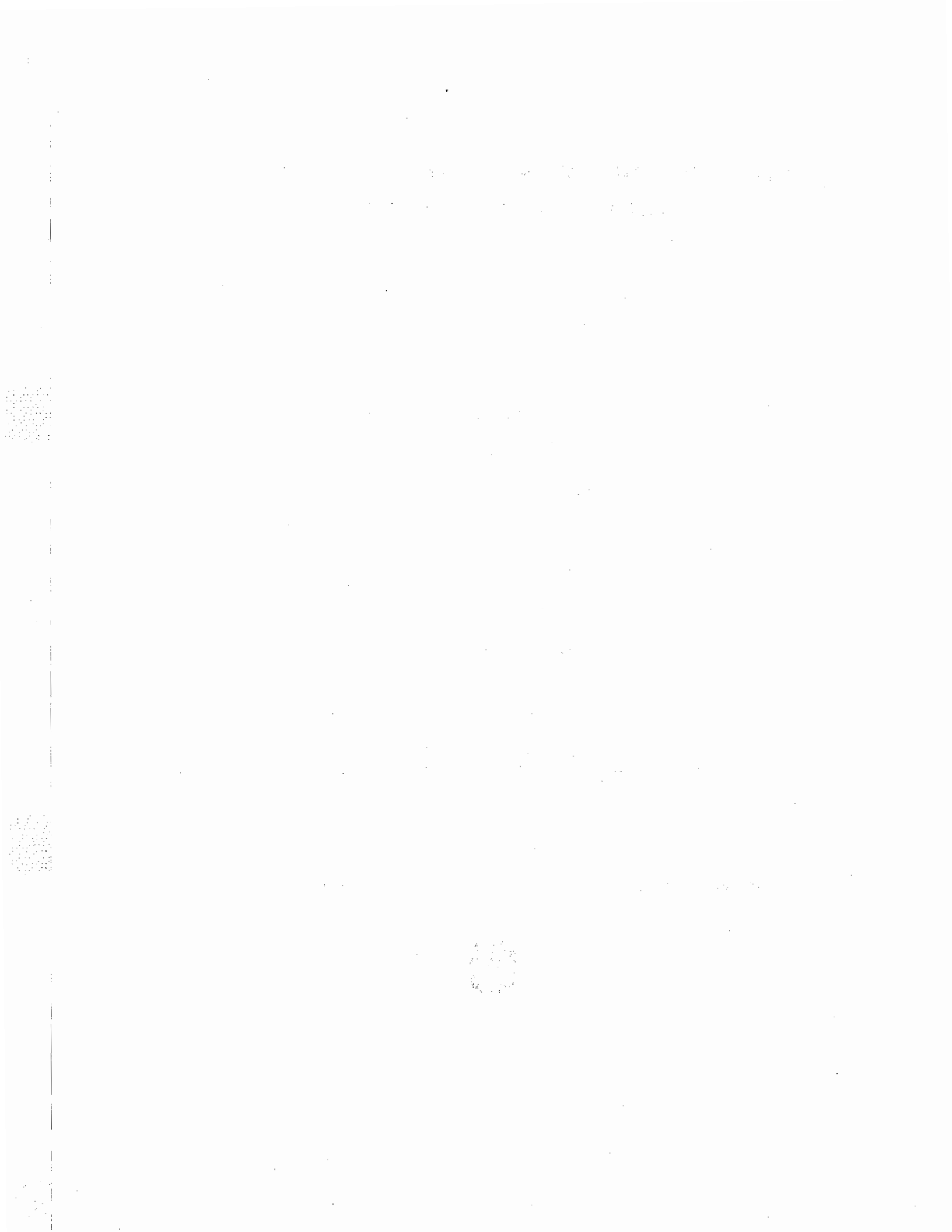
DYNAMIC SOIL PROPERTY TESTING AND ANALYSIS OF SOIL PROPERTIES

DAYLIGHT and McCUTCHANVILLE, INDIANA

by

PAUL F. HADALA

and

RICHARD W. PETERSON

JANUARY 1993

(Revised November 1993)

Conducted by:

Geotechnical Laboratory
U.S. Army Engineer Waterways Experiment Station
Vicksburg, MS

Sponsored by:

Office of Surface Mining, U.S. Department of the Interior
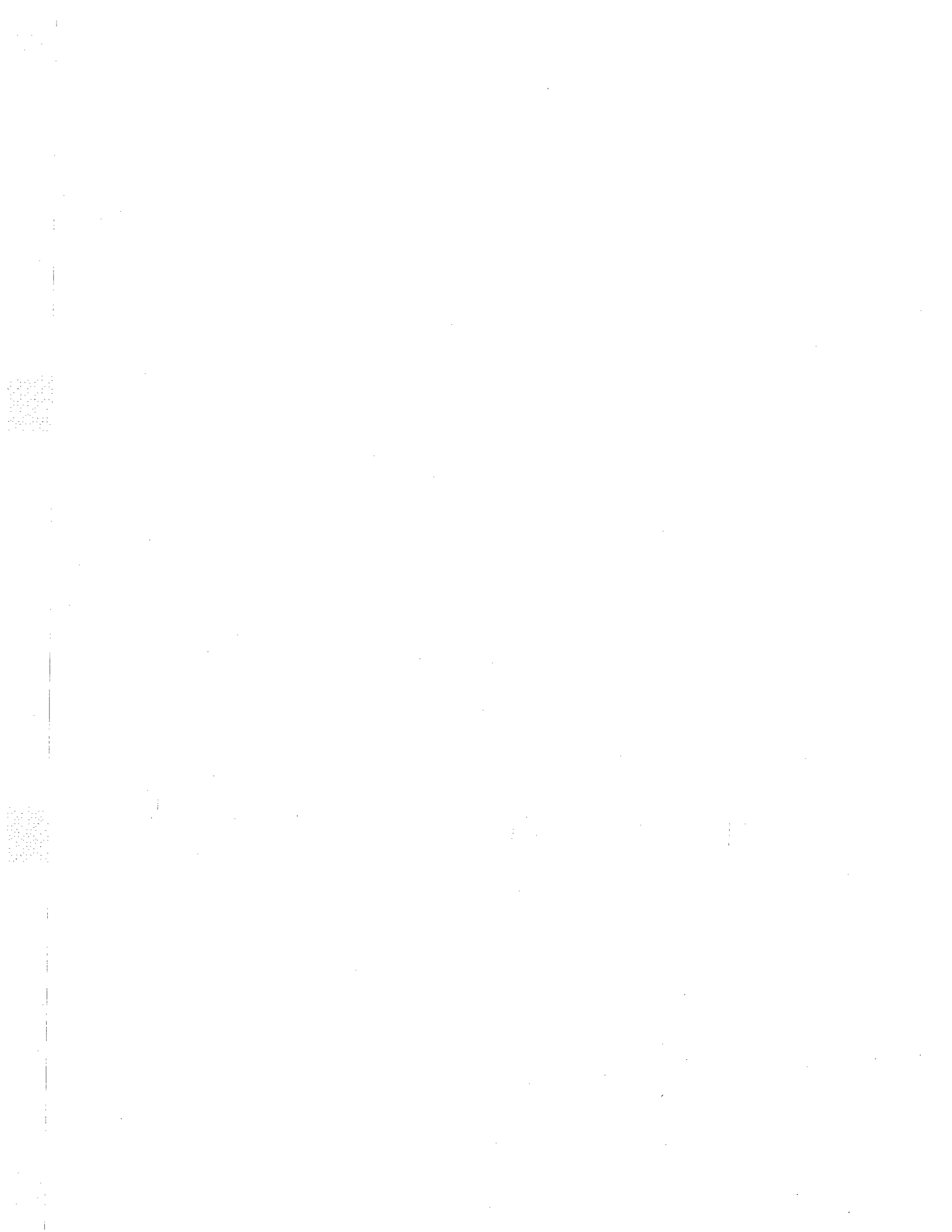
Revisions made in November 1993 include only pagination, pen and ink typographical corrections on pages 53 and 65, and the addition of Appendix B which was prepared in response to a telephone request for clarifications and additions by the sponsor in October 1993. A Table of Contents was also added.

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CHAPTER 1

Introduction

1. As part of a larger investigation of blasting related vibrations in Daylight and McCutchanville, Indiana, carried out by the Structures Laboratory (SL), U.S. Army Engineer Waterways Experiment Station (WES), the Geotechnical Laboratory (GL) of WES conducted supporting tests and analyses. The work was carried out for the Office of Surface Mining (OSM), Department of the Interior, under Interagency Agreement No. EF68-IA91-13796 during the period October 1991 to December 1992, and is related to concurrent tests and studies carried out by OSM and by the U.S. Bureau of Mines and the U.S. Geological Survey for OSM. The OSM technical monitor for this study was Mr. Peter Michael.

Objectives

2. The objectives of the GL studies were:

    a. To determine if undisturbed unsaturated soil samples from the Daylight/McCutchanville area could collapse under many cycles of low amplitude vibration.

    b. To determine if soil samples from the Daylight/McCutchanville area would experience an increase in pore pressure or loss of shear strength as a result of cyclic loading.

    c. To perform a static settlement analysis for a typical residential building foundation in the area. (Results of this analysis were given to SL for evaluation of the potential of the predicted settlement magnitudes to cause structural damage.)

    d. To perform a static earth pressure analysis for a typical residential basement wall. (Results of the analysis were also given to SL for evaluation of their potential for structural damage.)

Scope

3. Chapter 2 of this report is a Memorandum for Record addressing objectives 2.a. and 2.b. It was furnished to OSM in draft form on 17 Aug 92 and finalized on 30 Dec 92 based on review comments. Chapter 3 is a Memorandum for Record addressing objective 2.c. It was furnished to OSM in draft form on 18 Sep 92 and was finalized on 30 Dec 92 based on review comments. Chapter 4 is a Memorandum for Record addressing objective 2.d. It was furnished to OSM in draft form on 14 Jan 93 and finalized on 19 Jan 93 based on review comments.

4.   Appendix A is a trip report written by the senior author on 11 Mar 91 as a result of a reconnaissance visit made on 20-21 Feb 91 to observe a number of damaged buildings in the Daylight/McCutchanville area <u>before</u> the start of the present study.   It serves to document information available to the authors at the beginning of this study.   Because it is referenced extensively in some of the chapters and is not generally available, it has been included as an appendix.   At the request of the OSM technical monitor, the senior author revisited the hypothesis, conclusions, and judgments in this trip report in light of the additional information obtained during the course of the present study.   Chapter 4 of the main report also contains an addendum to Appendix A based on this additional information and identifies potential causal mechanisms for building damage not considered at the beginning of the study.

5.   This document serves only to collect and preserve the above mentioned memoranda under a single cover and place the documents in proper context with respect to one another and the project objectives.

CHAPTER 2

DYNAMIC SOIL PROPERTY TESTINGS

CEWES-GV-A   30 Dec 92

MEMORANDUM FOR RECORD

SUBJECT: Laboratory Soil Testing - Interagency Agreement No. EF68-IA91-13796,
"Field and Laboratory Evaluation of Potential Causative Factors of Structural
Damages in Daylight/McCutchanville, IN"

Introduction

1. Reference the section entitled "Statement of Work Laboratory Soil Testing"
of the subject Interagency Agreement (encl 1). The question addressed in this
study is: "Is there a potential for collapse of the structure of unsaturated
soils or for pore-pressure rise in saturated soils in the study area due to
ground vibration?" (quoted from page 2 of the Interagency Agreement). The
laboratory soils investigation to answer these questions was conducted by the
staff of the Geotechnical Laboratory (GL), U.S. Army Engineer Waterways Exper-
iment Station (WES) and is summarized in this memorandum.

2. On 24 May 92, 26 cardboard tube samples and 27 jar samples were delivered
to the GL Soils Humid Room for storage until the laboratory testing was con-
ducted. The information recorded on the boring logs was compared with the
data written on the identification tags for the samples. No inconsistencies
were found (see encl 2 for a copy of the boring logs). The samples were taken
by an OSM contractor with a 5-in. diameter fixed piston sampler supplied by
WES under the personal technical supervision of Mr. Mark Vispi of GL.

Laboratory Testing Program

3. Preliminary Test. On 26 May 92, one back-pressure saturated,
consolidated, undrained triaxial compression test with pore pressure
measurements was initiated. The test (number OSM-UD1-1-4.8) was conducted as
a preliminary test to determine whether the soil tended to expand or contract

ROUTING:
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| LABORATORY | LABORATORY | LABORATORY | LABORATORY | RESEARCH CENTER | TECHNOLOGY LABORATORY |

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in shear and to gain an understanding of the time required for consolidation.
Procedures and equipment used for this test were similar to those described in
EM 1110-2-1906[1] and ASTM Test Method D 4767[2]. The results of this test are
summarized in Table 1, "Summary of Triaxial Compression Test Results."

4. Cyclic Torsional Tests. The cyclic torsional test program was conducted
to evaluate the potential for collapse of the structure of unsaturated soils.
Dynamic low level cyclic torsional shear strain testing was initiated on
5 Jun 92. Fourteen tests were conducted on 2.8-in. diameter by 5.6-in. high
specimens in a Drnevich longitudinal-torsional free-fixed resonant column
apparatus. The equipment used for these tests was similar to the equipment
described in ASTM Standard Test Method D 4015[3]. A schematic of the testing
equipment is presented in Figure 1. Testing procedures consisted of a consol-
idation phase which was followed by a dynamic torsional shear phase for each
specimen. The consolidation phase consisted of the application of an isotro-
pic stress which was equivalent to the estimated in situ overburden stress for
that specimen. After the specimen had equilibrated under the applied stress,
the dynamic shear phase (with open drainage) was conducted. The shear phase
consisted of the application of a cyclic torsional shear stress to the
specimen to cause a desired amplitude of shear strain at the frequency of
interest. Generally, two amplitudes of shear strain (0.01 and 0.04 percent)
were applied to each specimen in a drained condition. The cyclic frequency
was 20 Hz (20 cycles per second). This frequency was chosen because it was
the lowest at which control of the apparatus could be maintained. It is on
the upper end of the range of frequencies measured in the field rather than in
the middle as would have been most desirable. Seventy-two thousand cycles at
0.01 percent shear strain were applied, then the specimen's vertical
deformation was monitored for two hours. An additional 72,000 cycles at
0.04 percent shear strain were applied and the specimen was monitored for
another two hours. The dynamic shear phase of each test took about six hours.
The axial deformation of each specimen was monitored throughout the dynamic
shear phase. Each specimen was tested at its "natural" or in situ water
content. The results of the cyclic torsional shear tests are summarized in
Table 2, "Summary of Dynamic Torsional Shear Test Results."

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[1] Engineer Manual EM 1110-2-1906, Department of the Army, Office of the
Chief of Engineers, "Laboratory Soils Testing," Washington, DC, 1970.

[2] American Society for Testing and Materials Standard Test Method D 4767,
"Consolidated-Undrained Triaxial Compression Test on Cohesive Soils," 1992
Annual Book of ASTM Standards, Volume 04.08, Soil and Rock; Dimension Stone;
Geosynthetics, Philadelphia, PA.

[3] American Society for Testing and Materials Standard Test Method D 4015,
"Modulus and Damping of Soils by the Resonant-Column Method."

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5. Shear strain levels used in these tests were selected as follows. A one-dimensional SH wave propagation model was assumed for the field condition
because it was mathematically tractable and known to give results within the
correct order of magnitude for three-dimensional explosion generated wave
propagation strain estimates. This model leads to a relation between peak
horizontal particle velocity and maximum shear strain.

$$v = C_s \epsilon$$

where v = peak particle velocity

C_s = shear wave velocity

ϵ = maximum shear strain

6. Figure 2 shows curves relating these parameters. The maximum measured
particle velocity near a complainant's residence[4] (albeit during limited
periods of observation) was about 0.10 in./sec. The shear wave velocity
indicated by the resonant column tests discussed in paras. 9 and 13.h. of this
memorandum ranged from 372 to 481 ft/sec. Shear velocities measured in the
near surface soils by the USGS ranged from 380 to 780 ft/sec. Based on these
data and the assumed model, the peak shear strain that is estimated to have
occurred in the field would be 0.002 percent. In order to conservatively
overcome any error in estimated strain associated with a) the assumed model,
or b) the possibility that a somewhat larger particle velocity occurred in
blasts where there was no monitoring equipment at complainant residences,
strain levels 5 and 20 times the predicted value were used in the laboratory
program.

7. The mine was in operation since 1973 with approximately six significant
shots per week, all with about ten significant cycles of vibration (indicated
in typical ground motion records for some of the shots). This represents
about 60,000 cycles of vibration. Again, to be conservative, 144,000 cycles
were applied.

8. The objective of the cyclic torsional shear tests was to determine if
there was any potential for the soil to collapse its structure under many

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[4] Siskind, D.E., Crum, Steven B., and Plis, Mathew N. "Vibration Envi-
ronment and Damage Characterization for Houses in McCutchanville and Daylight,
Indiana," Contract Research Report Feb. 1990, Department of the Interior,
Office of Surface Mining.

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cycles of strain at or above those experienced in the field. Collapse
potential was measured by monitoring the vertical deformation of the 5.6-in.
high confined specimens as cyclic torsional loading was applied. In terms of
maximum strain amplitude and total number of cycles, the environment created
in the laboratory was more severe than that which occurred in the field in the
Daylight and McCutchanville areas. Note also that the strain amplitudes and
number of cycles are both larger than that specified in the Interagency
Agreement (Section IV:B.1.f.) (encl 1). This change was made when it was
found in preliminary testing that the smaller amplitudes and durations gave
responses at or below our capability to measure vertical and torsional
displacement (0.0001 in.) in the laboratory environment.

9. Resonant Column Tests. Two of the specimens subjected to the cyclic
torsional shear tests described in paragraph 4 were tested as resonant column
tests at the conclusion of the dynamic torsional shear test. This was done in
order to obtain information on the shear wave velocity of the soils to
supplement the USGS field data. Use of the shear wave velocity data to
compute strain levels to be expected was discussed in para. 6. The testing
methods, equipment, and procedures were similar to the methodology described
in ASTM Standard Test Method D 4015[5]. The results of the resonant column
tests are summarized in Table 3, "Summary of Resonant Column Test Results."

10. Monotonic and Cyclic Triaxial Compression Tests. This group of tests was
conducted for the purpose of determining whether there was any loss of shear
strength of near surface soils due to cyclic loading and if there was any
potential for pore pressure generation by cyclic loading. Beginning 7 Jul 92,
triaxial compression tests (numbers OSM-UD1-2.6.1-1, OSM-UD1-2-6.1-2,
OSM-UD2-4-13.5-1, and OSM-UD-2-4-13.5-2) were conducted on four specimens
tested at their natural water contents (i.e., unsaturated. See para. 13.d).
Two of the specimens were consolidated and sheared monotonically (with
drainage open) using equipment and procedures similar to those described in
Engineer Manual EM 1110-2-1906 and ASTM Standard Test Method D 4767. The
procedures used during the shear phase for the other two specimens were
modified as follows. Following the consolidation phase, 20 cycles (at 1 Hz)
of axial dynamic deviator stress were applied (with drainage open) using the
stress controlled loading mode. When the cyclic loading was completed, each
specimen was subjected to strain controlled monotonic loading with drainage
open until failure (5 percent axial strain) occurred. During the dynamic
loading phase, the extension and compression loads applied to the specimens
were sufficiently large to ensure that a reversal of the major principal
stresses occurred. Cyclic reversal of principal stress has been shown to
generate pore pressure in some soils. The zero to peak amplitudes of the

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[5] "American Society for Testing Materials Standard Test Method D 4015,
"Modulus and Damping of Soils by the Resonant-Column Method."

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cyclic deviator stress were approximately 2 psi (this is 4 percent to
7 percent of the deviator stresses at failure under static load). The
resulting zero to peak cyclic axial strains were from 0.001 to 0.02 percent
and are respectively within and significantly above the range expected based
on the calculations in para. 6. The results of these tests are summarized in
Table 1, "Summary of Triaxial Compression Test Results."

11. A back pressure saturated, consolidated, undrained triaxial compression
test (number OSM-UD2-1-4.8-1) similar in most respects to the one described in
para. 3, and whose data are discussed in para. 13.e., was conducted 19-24 Aug
92 on a specimen from the Zimmermann property obtained at the same nominal
depth as the earlier test. The test was different from the earlier test
because an undrained cyclic loading phase was inserted following the
consolidation phase (in which the specimen was isotopically consolidated to
5 psi) and before the undrained shear phase of the test. The specimen was
subjected to 20 cycles of a 1 Hz cyclic deviator stress of ~2 psi about the
isotropic consolidation stress of 5 psi during this phase. Results of the
test are reported in Table 1.

12. Miscellaneous Tests. Selected index tests were also conducted on each
specimen. Atterberg limits were determined for all specimens. A specific
gravity test was conducted on the specimen which was used for the back
pressure saturated triaxial test (number OSM-UD1-1-4.8). It was believed that
one test was sufficient to index this property because the materials which
were encountered during the investigation were similar and the range of
specific gravities for most soils is fairly limited. The procedures and
equipment which were used for the Atterberg Limits tests and the specific
gravity test were described in Engineer Manual EM 1110-2-1906. Similar
procedures and equipment are also described by ASTM Standard Test
Methods D 4318, "Liquid Limit, Plastic Limit, and Plasticity Index of Soils",
and D 854, "Specific Gravity of Soils", respectively. The results of the
tests for each specimen are summarized in Tables 1 and 2.

13. Test Results and Observations. Pertinent observations based on the data
obtained during the investigation are summarized below:

    a. Most specimens were a mottled brown to gray silty clay with iron
oxide nodules. The size of the iron oxide nodules varied from very small to
about 1/4 in. diameter.

    b. The results of the Atterberg limits tests indicated the soil was a
low plasticity clay (CL). The liquid limits ranged from 45 percent to 22
percent and the plasticity indexes ranged from 25 percent to 7 percent.

    c. The specific gravity of the one specimen tested was 2.73.

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d. The test specimens were dense to very dense. Void ratios ranged from
0.79 to 0.52. The initial degrees of saturation of the specimens ranged from
80 percent to 98 percent, although the degrees of saturation for most of the
specimens ranged from 85 percent to 95 percent. The question quoted in para. 1
implies that some fully saturated samples would be encountered in the
undisturbed sampling program. As indicated by Tables 1 and 2, no saturated
samples were encountered in the samples taken which ranged in depth from 4 ft
to 14 ft.

e. The results of the back-pressure saturated, consolidated, undrained
triaxial compression test with pore pressure measurements indicated that the
artificially saturated specimen was dense and strong. Based upon the stress
path data, failure occurred at an axial strain of 5 percent to 6 percent. The
effective angle of internal friction was 30 degrees and the cohesion intercept
was about 2 psi. An effective angle of internal friction of 30 degrees for a
clay soil is fairly large. Skempton's A-pore pressure parameter was about
-0.8. This value indicated that the specimen tended to dilate strongly during
shear and inferred that the specimen was highly overconsolidated. It also
infers that the hypothesis that positive pore pressure development and
strength reduction due to cyclic loading is unlikely. The test results are
summarized as a deviator stress versus axial strain relationship in Figure 3,
as an induced pore pressure versus axial strain relationship in Figure 4, and
as a shear stress versus normal stress relationship in Figure 5.

f. The back pressure saturated, consolidated, cyclic, and monotonic
loading undrained triaxial compression test which was a companion to that
discussed in para. 13.e. did not result in any residual pore pressure mobili-
zation at the ends of the specimen after cyclic loading. However, there was a
small pore pressure oscillation of ~0.4 psi during the cycling of the deviator
stress. The specimen tried to dilate during the subsequent monotonic loading.
Skempton's A-pore pressure parameter during monotonic shear was about -0.1.
The maximum deviator stress during shear was 8.4 psi versus 12.8 psi in the
uncycled specimen. However, the initial and post consolidation void ratios of
the cyclically loaded specimen were substantially higher than those of its
companion specimen. This void ratio difference fully accounts for the
strength differences. Test results are plotted in Figures 3, 4, and 5 for
comparison with the non-cycled companion test.

g. The results of the dynamic torsional shear tests indicated there was
little tendency for axial deformation to occur during shear. Typically the
axial strains which were caused by torsional loading were less than 0.02
percent. After the shear phase was completed, most specimens rebounded to
the height of the specimen prior to the shear test. This behavior may be
described as viscoelastic, as compared to the plastic behavior which occurred
during the consolidation phase for each test specimen.

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h. Two specimens were tested as resonant column tests after the dynamic
shear tests (see paras. 6 and 9) were completed. Shear strains ranged from
less than 0.001 percent to about 0.02 percent. The corresponding maximum
shear moduli were of the order of 5000 psi, but decreased to about 3000 psi at
larger shear strains. Material damping increased from 3 percent at smaller
shear strains to about 7 percent at larger shear strains (see Table 3).

i. The results of the consolidated drained triaxial compression tests on
specimens tested at their natural water contents (i.e. unsaturated) indicated
that the application of 20 cycles of dynamic axial loading did not affect the
consolidated drained strength of the specimens significantly. The test
results were expressed as deviator stress versus axial strain relationships in
Figure 6. For the specimens from a depth of 6.1 ft, the deviator stress at
failure for the specimen which was loaded cyclicly prior to monotonic shear
was 31 psi, while the deviator stress for the specimen subjected only to
monotonic shear was 30 psi. For specimens from a depth of 13.5 ft, the
deviator stresses at failure were about 50 psi. The test data were compared
at an axial strain of 5 percent. This value of strain corresponded to the
axial strain at which the back pressure saturated triaxial compression test
specimen failed. The shear strengths were also compared at axial strains of
about 15 percent. For the specimens from a depth of 6.1 ft, the deviator
stress for the specimen loaded cyclicly prior to monotonic shear was about
34 psi, while the deviator stress for the specimen subjected to only monotonic
shear was about 28 psi. For the specimens from a depth of 13.5 ft, the
deviator stress for the specimen which was loaded cyclicly prior to monotonic
shear was about 49 psi while the deviator stress for the specimen which was
subjected to only monotonic shear was about 55 psi. The differences noted are
not large enough to cause foundation instability and are explainable by the
small differences in initial water content and void ratios between companion
specimens.

Analysis and Conclusions (underlined)

14. Based upon the analysis of the data obtained from the laboratory tests,
no anomalies were identified. The responses of the specimens to the loading
conditions were anticipated. The material was a dense to very dense silty
clay. It exhibited a fairly large angle of internal friction. Under the
specified loading conditions, the specimens subjected to smaller shear strains
responded "visco-elastically" whereas the specimens subjected to larger shear
strains deformed "plastically." The latter group of specimens tended to
dilate during shear. The laboratory data inferred that the soil would perform
adequately as a foundation material for lightly loaded structures.

15. Under the sustained oscillating shear strain environment created in the
torsional shear tests, the six in. high specimens changed in height by an
amount ranging from 5 to 15 ten-thousandths of an inch. This is a vertical

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strain of 0.025 percent or less. Even if such strain occurred uniformly over
a 100-ft deep soil column, only 0.3 in. of surface displacement would result.
Differential displacement between two surface points would be less. Con-
ventional residential structures would not be damaged by these conditions.
The torsional shear tests conducted on samples from three different sites
offered no evidence to suggest the existence of any kind of collapse mechanism
or creep mechanism caused or triggered by sustained low level vibration. The
specimens essentially behaved visco-elastically in the tests conducted. This
eliminates soil structure collapse or creep due to sustained vibration from
the list of possible causal mechanisms for the observed building distress.

16. The comparison of the consolidated drained cyclic and non-cyclic triaxial
tests showed no strength loss due to cycling of the deviator stress at
4 percent to 7 percent of the peak strength. These tests were on unsaturated
specimens and, because of this, shed no light on the question of whether
positive pore pressure would have been mobilized by cycling a saturated
undrained specimen. The consolidated undrained static test was performed on
an artificially saturated specimen. It indicated negative pore pressure in
shear at large shear strains. Negative pore pressures increase rather than
degrade strength. One consolidated undrained test (with cyclic loading) was
also performed on an artificially saturated specimen. It showed:

     a. No induced pore pressure at the ends of the specimen after cyclic
loading.

     b. A tendency to dilate during shear as inferred by the negative pore
pressure response during undrained shear.

     c. A shear strength consistent with the shear strength of the uncycled
test when the initial, naturally occurring void ratio differences between the
specimens are considered.

The fact that all specimens from depths of 14.0 ft or less for which degrees
of saturation in situ were determined were less than 98 percent, and most were
less than 95 percent, and all of the trends exhibited in the triaxial testing
program indicate that positive pore pressures induced by cyclic undrained
loading are just not possible in the in situ condition at depths sufficiently
shallow so as to impact the bearing capacity of residential building
foundations.
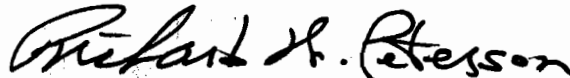
17. At the outset of this study, it was judged that the two hypothesized
causal mechanisms (pore pressure rise or collapse under cyclic loading) to be
evaluated in the laboratory dynamic soil testing programs were unlikely.
Nevertheless since it was possible to conduct definitive laboratory testing
which would eliminate the need for judgment, it was decided to proceed.

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<u>The testing program confirmed that these hypothesized causal mechanisms were
not physically possible in the soils tested.</u>

18. If you have questions regarding the laboratory test results or the inter-
pretation of the data, please call Dr. Richard W. Peterson at 601-634-3737 or
Dr. Paul F. Hadala at 601-634-3475.

RICHARD W. PETERSON   PAUL F. HADALA
Research Civil Engineer   Assistant Director
Geotechnical Laboratory   Geotechnical Laboratory

11 Encls

CF:
Mr. Peter Michael, Office of Surface Mining
Mr. Vince Chiarito, CEWES-SS-A

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Table 1. Summary of Triaxial Compression Test Results

Site	Test Number	Test Date(s)	Boring Number	Sample Number	Depth ft.	Atterberg Limits LL %	PL %	PI %	Specific Gravity	Height in.	Diam- eter in.	Initial Conditions Water Content %	Dry Density pcf	Void Ratio	Satu- ration %	Saturation* Stress Difference psi	Height Change in.
Preliminary Back Pressure Saturated Consolidated Undrained Test																	
Zimmerman	OSM-UD1-1-4.8	28 May - 8 Jun 92	UD1	1	4.8	45	20	25	2.73	3.0	1.374	25.1	100.4	0.697	98.2	3	0.009
Comparison Back Pressure Saturated Consolidated Undrained Test with Cycling																	
Zimmerman	OSM-UD2-1-4.8-1	19-24 Aug 92	UD2	1	4.8	42	18	24	----	6.044	2.788	26.1	95.8	0.779	91.3	2	-0.008
Consolidated-Drained Tests on Unsaturated Specimens Without (-2 at end of test no) and With (-1 at end of test no) Cycling																	
Zimmerman	OSM-UD1-2-6.1-1	7-10 Jul 92	UD1	2	6.1	35	19	16	----	5.616	2.745	25.1	98.0	0.738	92.9	---	---
Zimmerman	OSM-UD1-2-6.1-2	7-8 Jul 92	UD1	2	6.1	35	19	16	----	2.994	1.368	24.3	100.3	0.699	94.9	---	---
Zimmerman	OSM-UD2-4-13.5-1	13-16 July 92	UD2	4	13.5	34	17	17	----	6.203	2.766	18.9	107.0	0.593	86.9	---	---
Zimmerman	OSM-UD2-4-13.5-2	9-10 Jul 92	UD2	4	13.5	34	17	17	----	3.006	1.366	19.1	105.7	0.612	85.0	---	---

* Volumetric strains were assumed to be isotropic.

\# Maximum difference between confining stress and back pressure permitted during saturation.

Table 1. Summary of Triaxial Compression Test Results (continued)

| Site | Test Number | Consolidation Phase | | | | | | | | | Shear Phase | | | | | | |
		Effective Confining Pressure psi	Height Change in.	Axial Strain %	Volume Change cm³	Volume Strain %	Radial Strain %	Time t₅₀ min	t₁₀₀ min	Void Ratio	Dev Stress+ psi	Axial Strain %	A**	Stress Path Data Intercept psi	Slope	Cohesion c' psi	Friction φ' deg
Preliminary Back Pressure Saturated Consolidated Undrained Test																	
Zimmerman	OSM-UD1-1-4.8	5	0.005	0.167	0.9	1.235	0.534	31	480	0.641	12.8	5.0	-0.8	1.3	0.5	2	30
Comparison Back Pressure Saturated Consolidated Undrained Test with Cycling																	
Zimmerman	OSM-UD2-1-4.8-1	5	0.011	0.182	3.7	0.608	0.213	1#	10#	0.775	8.4	5.0	-0.1	1.1	0.4	1	21
Consolidated-Drained Tests on Unsaturated Specimens Without (-2 at end of test no) and With (-1 at end of test no) Cycling																	
Zimmerman	OSM-UD1-2-6.1-1	6	0.006	0.107	--	-----	-----	--	--	0.732*	31.4	5.3	----	---	---	--	--
Zimmerman	OSM-UD1-2-6.1-2	6	0.007	0.224	--	-----	-----	--	--	0.688*	29.4	5.3	----	---	---	--	--
Zimmerman	OSM-UD2-4-13.5-1	13	0.019	0.306	--	-----	-----	--	--	0.578*	50.4	5.4	-----	---	---	--	--
Zimmerman	OSM-UD2-4-13.5-2	14	0.009	0.296	--	-----	-----	--	--	0.598*	50.5	5.1	-----	---	---	--	--

* Volumetric strains were assumed to be isotropic.
+ Failure was assumed to occur at about 5 percent axial strain, based upon the stress path data for the saturated specimens.
# Axial and radial drainage was allowed during consolidation.
** Skerpton's A pore pressure parameter.

Table 1. Summary of Triaxial Compression Test Results (concluded)

Site	Test Number	Remarks
Preliminary Back Pressure Saturated Consolidated Undrained Test		
Zimmerman	OSM-UD1-1-4.8	Brown silty clay (CL) with 1/4 inch iron oxide nodules. Specimen was tested as a back-pressure saturated, consolidated, undrained triaxial compression test with pore pressure measurements. B-pore pressure parameter was greater than 0.95 but was not recorded.
Comparison Back Pressure Saturated Consolidated Undrained Test with Cycling		
Zimmerman	OSM-UD2-1-4.8-1	Brown silty clay (CL) with softer gray clay lenses and iron oxide nodules. Specimen was tested as a back-pressure saturated, consolidated, undrained triaxial compression test with pore pressure measurements. Specimen was loaded cyclicly prior to monotonic shear. B-pore pressure parameter was 0.95.
Consolidated-Drained Tests on Unsaturated Specimens Without (-2 at end of test no) and With (-1 at end of test no) Cycling		
Zimmerman	OSM-UD1-2-6.1-1	Brown silty clay (CL) with softer gray clay lenses and iron oxide nodules. Specimen was loaded cyclicly prior to monotonic shear.
Zimmerman	OSM-UD1-2-6.1-2	Brown silty clay (CL) with softer gray clay lenses and iron oxide nodules. Specimen was loaded monotonically to failure.
Zimmerman	OSM-UD2-4-13.5-1	Brown silty clay (CL) with iron oxide nodules. Specimen was loaded cyclicly prior to monotonic shear.
Zimmerman	OSM-UD2-4-13.5-2	Brown silty clay (CL) with iron oxide nodules. Specimen was loaded monotonically to failure.

Table 2. Summary of Dynamic Torsional Shear Test Results

Site	Test Number	Test Date(s)	Boring Number	Sample Number	Depth ft.	Initial Conditions						Final Conditions					
						Height in.	Diameter in.	Water Content %	Dry Density pcf	Void Ratio	Saturation %	Height in.	Diameter in.	Water Content %	Dry Density pcf	Void Ratio	Saturation %
Zimmerman	OSM-UD1-1-4.4	4-5 Jun 92	UD1	1	4.4	5.543	2.752	25.5	96.4	0.769	90.5	5.491	2.741	24.6	98.1	0.738	91.0
Zimmerman	OSM-UD1-2-6.8	8-9 Jun 92	UD1	2	6.8	5.549	2.739	25.5	98.5	0.730	95.4	5.546	2.729	25.0	99.3	0.717	95.4
Zimmerman	OSM-UD2-2-6.0	26-27 Jun 92	UD2	2	6.0	5.555	2.770	21.9	103.5	0.647	92.2	5.555	2.756	21.4	104.5	0.630	92.8
Zimmerman	OSM-UD2-4-14.1	29-30 Jun 92	UD2	4	14.1	5.578	2.768	20.8	104.2	0.636	89.5	5.574	2.764	20.5	104.6	0.630	89.1
Zimmerman	OSM-UD3-1-4.9	9-10 Jun 92	UD3	1	4.9	5.522	2.735	26.8	92.5	0.844	86.6	5.442	2.714	25.4	95.3	0.789	87.8
Zimmerman	OSM-UD3-3-13.2	10-11 Jun 92	UD3	3	13.2	5.513	2.806	24.2	100.2	0.701	94.3	5.474	2.796	23.2	101.6	0.677	93.7
Kansas Rd	OSM-UD4-1-5.0	12-13 Jun 92	UD4	1	5.0	5.512	2.810	18.2	106.8	0.596	83.2	5.502	2.800	18.1	107.5	0.585	84.2
Kansas Rd	OSM-UD4-2-6.3	17-18 Jun 92	UD4	2	6.3	5.553	2.842	20.6	105.6	0.615	91.6	5.548	2.837	20.3	106.0	0.607	91.1
Kansas Rd	OSM-UD5-2-10.7	15-16 Jun 92	UD5	2	10.7	5.488	2.792	20.4	108.3	0.574	96.8	5.482	2.790	20.2	108.6	0.570	96.6
Kansas Rd	OSM-UD6-1-5.0	16 Jun 92	UD6	1	5.0	-----	-----	20.2	----	-----	----	-----	-----	----	----	-----	----
Kansas Rd	OSM-UD6-3-9.7	16-17 Jun 92	UD6	3	9.7	5.502	2.798	18.2	112.2	0.519	95.7	5.494	2.805	18.0	111.8	0.524	93.6
Riney	OSM-UD7-1-4.7	19-20 Jun 92	UD7	1	4.7	5.578	2.772	18.8	106.2	0.605	85.0	5.566	2.766	18.4	106.9	0.594	84.7
Riney	OSM-UD8-1-4.8	22-23 Jun 92	UD8	1	4.8	5.538	2.820	17.2	108.2	0.576	81.3	5.538	2.819	16.9	108.2	0.575	80.2
Riney	OSM-UD8-2-5.7	23-24 Jun 92	UD8	2	5.7	5.538	2.757	16.8	109.6	0.555	82.8	5.532	2.754	16.5	110.0	0.549	82.1
Riney	OSM-UD8-2-6.0	23 Jun 92	UD8	2	6.0	-----	-----	18.0	----	-----	----	-----	-----	----	----	-----	----
Riney	OSM-UD10-2-3.5	25-26 Jun 92	UD10	2	3.5	5.566	2.775	20.6	102.4	0.664	84.4	5.551	2.763	20.3	103.5	0.646	85.7

Table 2. Summary of Dynamic Torsional Shear Test Results (continued)

Summary of Test Data

Site	Test Number	Atterberg Limits			Specific Gravity	Isotropic Consolidation Phase				Dynamic Shear Phase					After 2 hr Rest		Overall Change of Dimensions	
		LL %	PL %	PI %		Consolidation Stress psi	Stress Change* psi	Height Change in.	Height in.	Frequency Hz.	Number Cycles	Shear Strain+ %	Height Change in.	Height in.	Height Change in.	Height in.	Axial Strain %	Radial Strain %**
Zimmerman	OSM-UD1-1-4.4	43	19	24	----	5	2	0.0070	5.536	20 / 20	72,000 / 72,000	0.010 / 0.033	-0.0002 / -0.0016	5.536 / 5.538	-0.0003 / -0.0005	5.537 / 5.539	0.94	0.40
Zimmerman	OSM-UD1-2-6.8	36	19	17	----	7	4	0.0092	5.540	20 / 20	72,000 / 72,000	0.010 / 0.040	0.0005 / 0.0010	5.539 / 5.538	-0.0002 / -0.0006	5.539 / 5.539	0.05	0.37
Zimmerman	OSM-UD2-2-6.0	33	17	16	----	7	4	0.0076	5.547	20 / 20	72,000 / 102,000	0.009 / 0.041	0.0003 / 0.0010	5.547 / 5.546	-0.0003 / -0.0009	5.547 / 5.547	0.00	0.50
Zimmerman	OSM-UD2-4-14.1	38	18	20	----	14	11	0.0149	5.563	20 / 20	72,000 / 72,000	0.010 / 0.040	0.0008 / 0.0014	5.562 / 5.562	-0.0008 / -0.0008	5.563 / 5.563	0.07	0.14
Zimmerman	OSM-UD3-1-4.9	43	19	24	----	6	3	0.0131	5.509	20 / 20	72,000 / 72,000	0.009 / 0.039	0.0004 / 0.0020	5.508 / 5.507	-0.0001 / -0.0005	5.508 / 5.507	1.45	0.77
Zimmerman	OSM-UD3-3-13.2	38	18	20	----	6	3	0.0160	5.497	20 / 20	72,000 / 72,000	0.010 / 0.042	0.0003 / -0.0009	5.497 / 5.497	-0.0001 / -0.0003	5.497 / 5.498	0.71	0.36
Kansas Rd	OSM-UD4-1-5.0	32	14	18	----	5	2	0.0056	5.506	20 / 20	72,000 / 72,000	0.010 / 0.042	0.0011 / 0.0014	5.505 / 5.504	-0.0005 / -0.0013	5.506 / 5.506	0.18	0.36
Kansas Rd	OSM-UD4-2-6.3	37	17	20	----	6	3	0.0053	5.548	20 / 20	72,000 / 72,000	0.010 / 0.042	0.0007 / 0.0013	5.547 / 5.546	-0.0007 / -0.0012	5.548 / 5.548	0.09	0.18
Kansas Rd	OSM-UD5-2-10.7	40	19	21	----	4	1	0.0018	5.486	20 / 20	72,000 / 72,000	0.010 / 0.043	0.0002 / 0.0003	5.486 / 5.485	-0.0005 / -0.0011	5.486 / 5.486	0.11	0.07
Kansas Rd	OSM-UD6-1-5.0	25	18	7	----	--	--	------	-----	--	------	------	-------	-----	-------	-----	---	---
Kansas Rd	OSM-UD6-3-9.7	34	17	17	----	3	0	0.0006	5.501	20 / 20	72,000 / 72,000	0.011 / 0.041	0.0003 / 0.0006	5.501 / 5.500	-0.0002 / -0.0015	5.501 / 5.502	0.15	0.25
Riney	OSM-UD7-1-4.7	25	17	8	----	4	1	0.0040	5.574	20 / 20	72,000 / 72,000	0.010 / 0.040	0.0008 / 0.0008	5.574 / 5.573	-0.0002 / -0.0008	5.574 / 5.574	0.22	0.22
Riney	OSM-UD8-1-4.8	22	14	8	----	5	2	0.0085	5.530	20 / 20	72,000 / 72,000	0.009 / 0.042	0.0013 / 0.0016	5.528 / 5.528	-0.0010 / -0.0015	5.529 / 5.529	0.00	0.04
Riney	OSM-UD8-2-5.7	24	15	9	----	6	3	0.0048	5.533	20 / 20 / 20	72,000 / 72,000 / 144,000	0.010 / 0.040 / 0.041	0.0012 / 0.0012& / 0.0013	5.532 / 5.532 / 5.531	-0.0008 / -0.0011 / -0.0010	5.533 / 5.533 / 5.532	0.11	0.11
Riney	OSM-UD8-2-6.0	30	16	14	----	--	--	------	-----	--	------	------	-------	-----	-------	-----	---	---
Riney	OSM-UD10-2-3.5	30	18	12	----	4	1	0.0033	5.563	20 / 20	72,000 / 72,000	0.010 / 0.040	0.0004 / 0.0012	5.562 / 5.561	-0.0004 / -0.0010	5.563 / 5.562	0.12	0.43

* The stress change is the difference between pressures which were applied to the specimens as the device was assembled (usually 3 psi) and the consolidation pressures.

+ This value is the single amplitude shear strain.

& A change of LVDT readings occurred during the dynamic shear test; this value was estimated.

** This includes the effect of the total sequence of testing (i.e. consolidation-vibration-rest-vibration-rest). Most of the change is due to the consolidation phase.

Table 2. Summary of Dynamic Torsional Shear Test Results (concluded)

Site	Test Number	Remarks
Zimmerman	OSM-UD1-1-4.4	Brown silty clay (CL) with iron oxide nodules. There was a shift of the LVDT reading when the pulse generator was turned on. A vacuum of approximately 14 psi was applied to specimen during the assembly of test apparatus. Vacuum was then reduced to zero and a chamber pressure of about 3 psi was applied the specimen. The specimen was permitted to equilibrate for about two hours before the consolidation stress was applied. The change of height of the specimen under 14 psi was not recorded.
Zimmerman	OSM-UD1-2-6.8	Brown silty clay (CL) with iron oxide nodules. Lower quadrant of specimen was a slightly softer, gray silty clay. A small amount of free water was in pore pressure lines at end of test.
Zimmerman	OSM-UD2-2-6.0	Brown silty clay (CL) with iron oxide nodules. Incorrect depth calculation; σ_c was small.
Zimmerman	OSM-UD2-4-14.1	Brown silty clay (CL) with iron oxide nodules.
Zimmerman	OSM-UD3-1-4.9	Brownish-gray silty clay (CL) with iron oxide nodules. A small amount of free water was in pore pressure lines at end of test.
Zimmerman	OSM-UD3-3-13.2	Brown silty clay (CL) with iron oxide nodules. Incorrect depth calculation; σ_c was too small.
Kansas Rd	OSM-UD4-1-5.0	Specimen description was not written on test data sheets.
Kansas Rd	OSM-UD4-2-6.3	Brown silty clay (CL) with iron oxide nodules.
Kansas Rd	OSM-UD5-2-10.7	Brown silty clay (CL) with iron oxide nodules. Incorrect depth calculation; σ_c was too small.
Kansas Rd	OSM-UD6-1-5.0	Brown silty clay (CL) with 1/2 inch iron oxide nodules. Could not trim specimen; no test.
Kansas Rd	OSM-UD6-3-9.7	Brown silty clay (CL) with iron oxide nodules. Incorrect depth calculation; σ_c was too small.
Riney	OSM-UD7-1-4.7	Brown silty clay (CL) with 1/2 inch iron oxide nodules. Incorrect depth calculation; σ_c was too small.
Riney	OSM-UD8-1-4.8	Brown silty clay (CL) with iron oxide nodules.
Riney	OSM-UD8-2-5.7	Brown silty clay (CL) with iron oxide nodules. LVDT shifted during 2nd shear phase; ran 3rd shear phase.
Riney	OSM-UD8-2-6.0	Brown silty clay (CL) with 2-1/2 inch sandstone cobbles. Could not trim specimen; no test.
Riney	OSM-UD10-2-3.5	Brown silty clay (CL) with iron oxide nodules.

Table 3.   Summary of Resonant Column Test Results

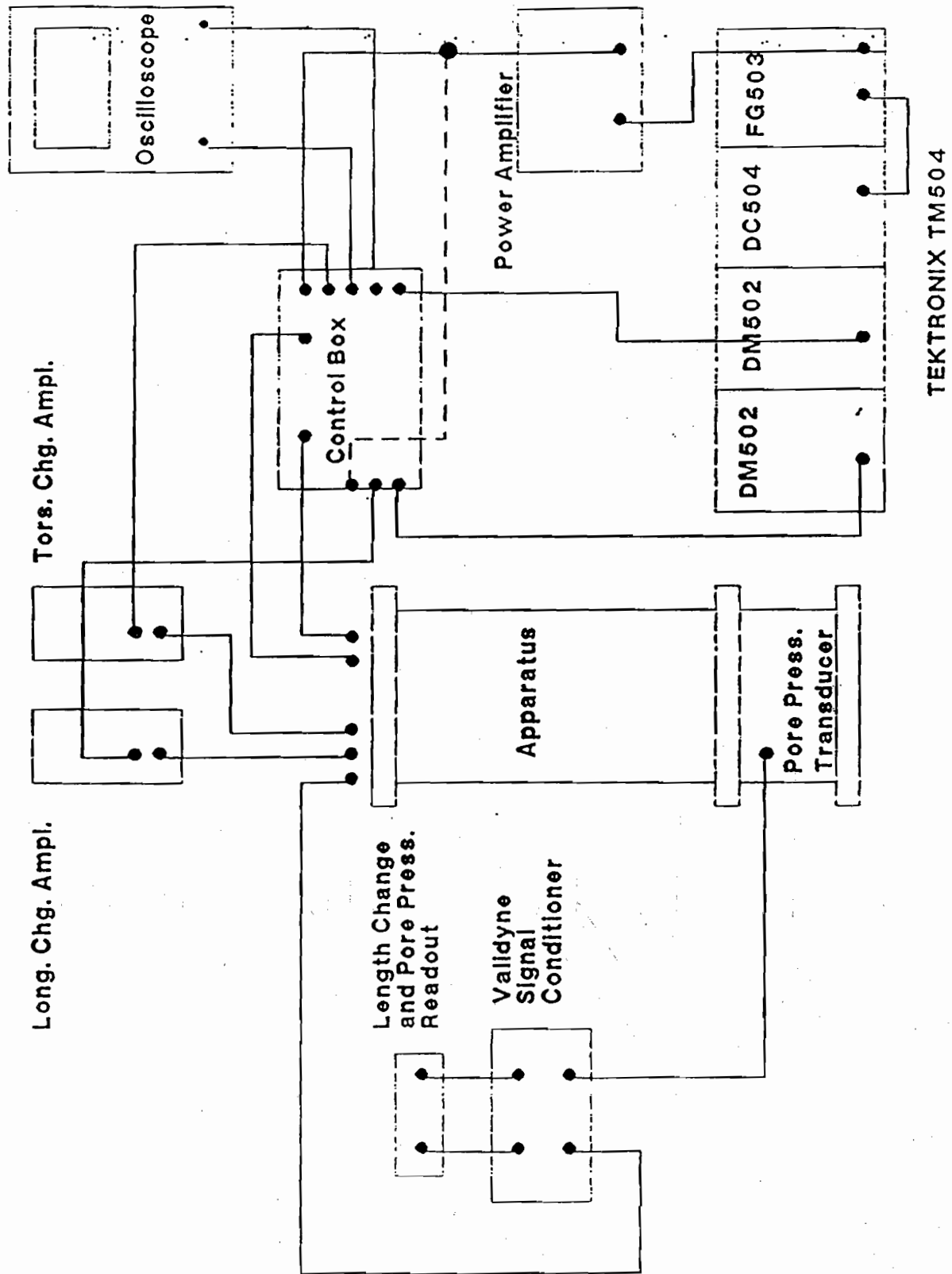| Site | Test Number | Resonant Frequency Test Results | | | Remarks |
		Shear Modulus psi	Shear Strain %	Damping Ratio %	
Zimmerman	OSM-UD1-1-4.4	4038	0.00083	3.1	Specimen was subjected
		3778	0.00162	4.3	to the dynamic shear
		3526	0.00244	4.8	test which was de-
		3403	0.00631	5.0	scribed in Paragraphs
		2823	0.01251	8.1	4 and 8f and Table 2
		2823	0.01255	7.7	prior to the Resonant Column Test.
Zimmerman	OSM-UD1-2-6.8	5624	0.00050	2.9	Specimen was subjected
		5475	0.00097	3.0	to the dynamic shear
		5183	0.00196	3.8	test which was de-
		4760	0.00524	4.0	scribed in Paragraphs
		4224	0.01323	4.7	4 and 8f and Table 2
		3843	0.01709	5.9	prior to the Resonant Column Test.

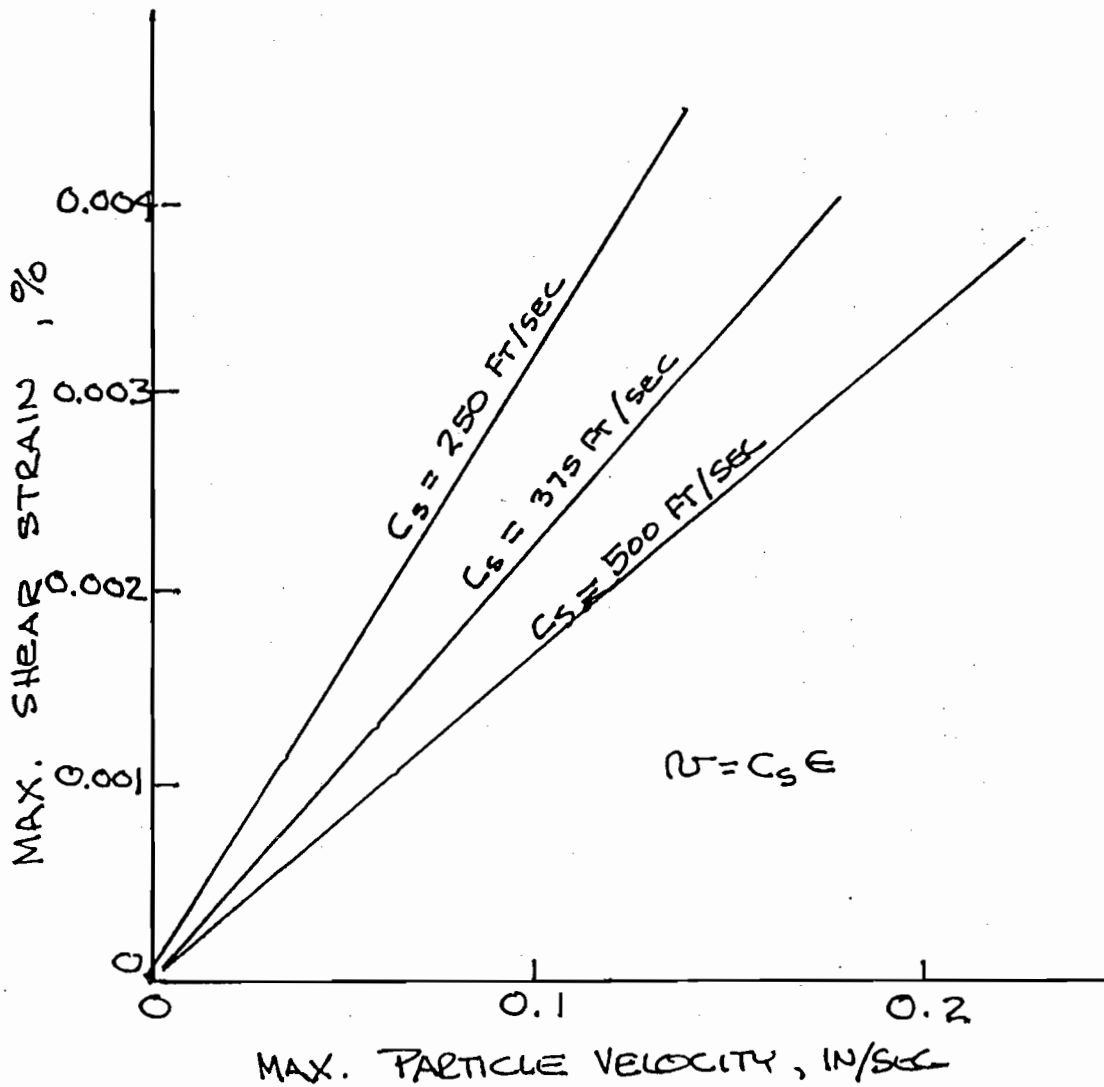Figure 1.   Resonant column apparatus.

Figure 2.   Shear strain versus particle velocity relationship for one-dimensional linear elastic wave propagation.

Figure 3. Deviator stress versus strain curves for consolidated undrained triaxial tests with and without low amplitude undrained cyclic loading following the consolidation phase.
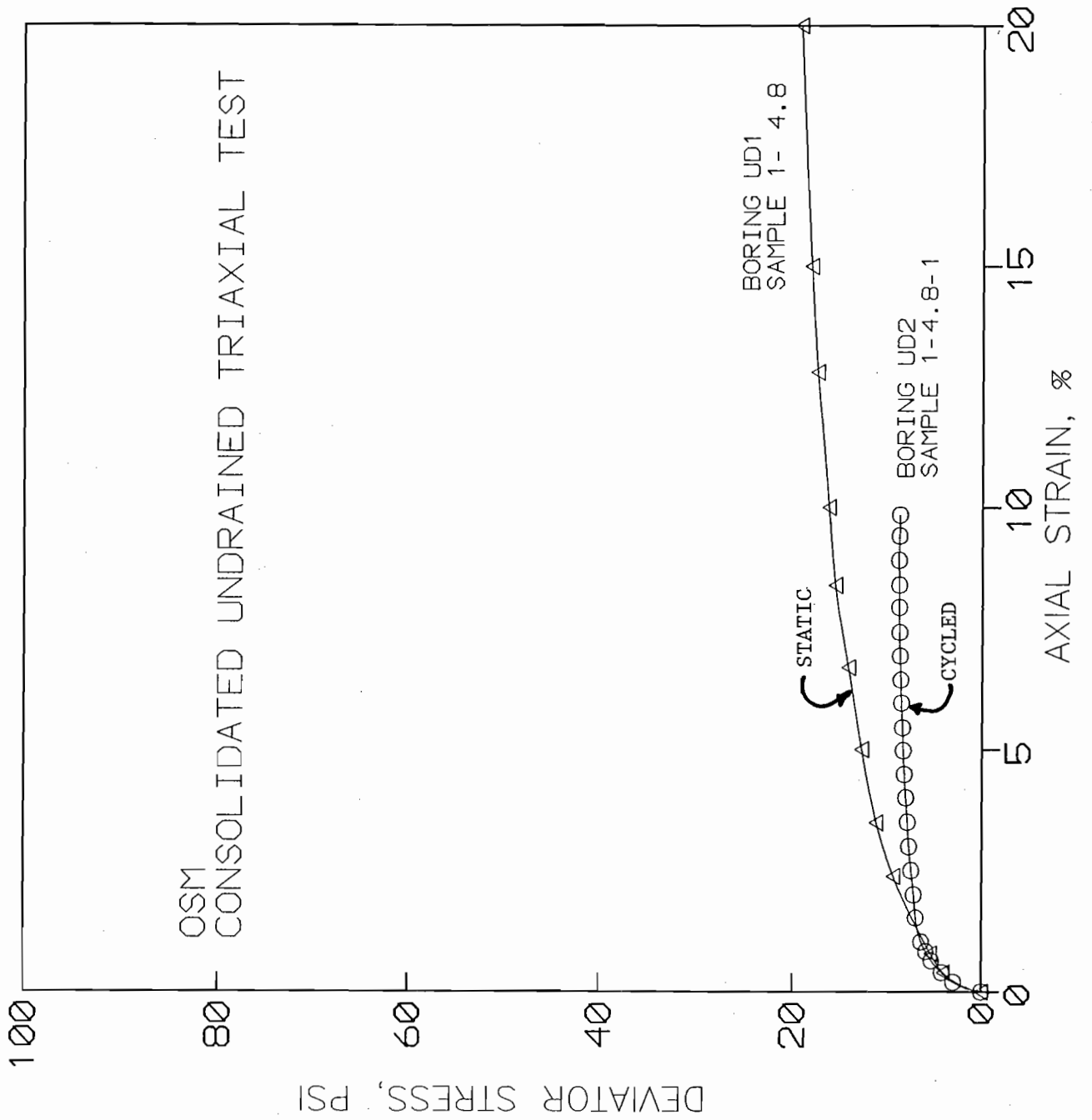
Figure 4.   Pore pressure versus strain curves for consolidated undrained
            triaxial tests with and without low amplitude undrained cyclic
            loading following the consolidation phase.

Figure 5. Stress paths for consolidated undrained triaxial tests with and without low amplitude undrained cyclic loading following the consolidation phase.
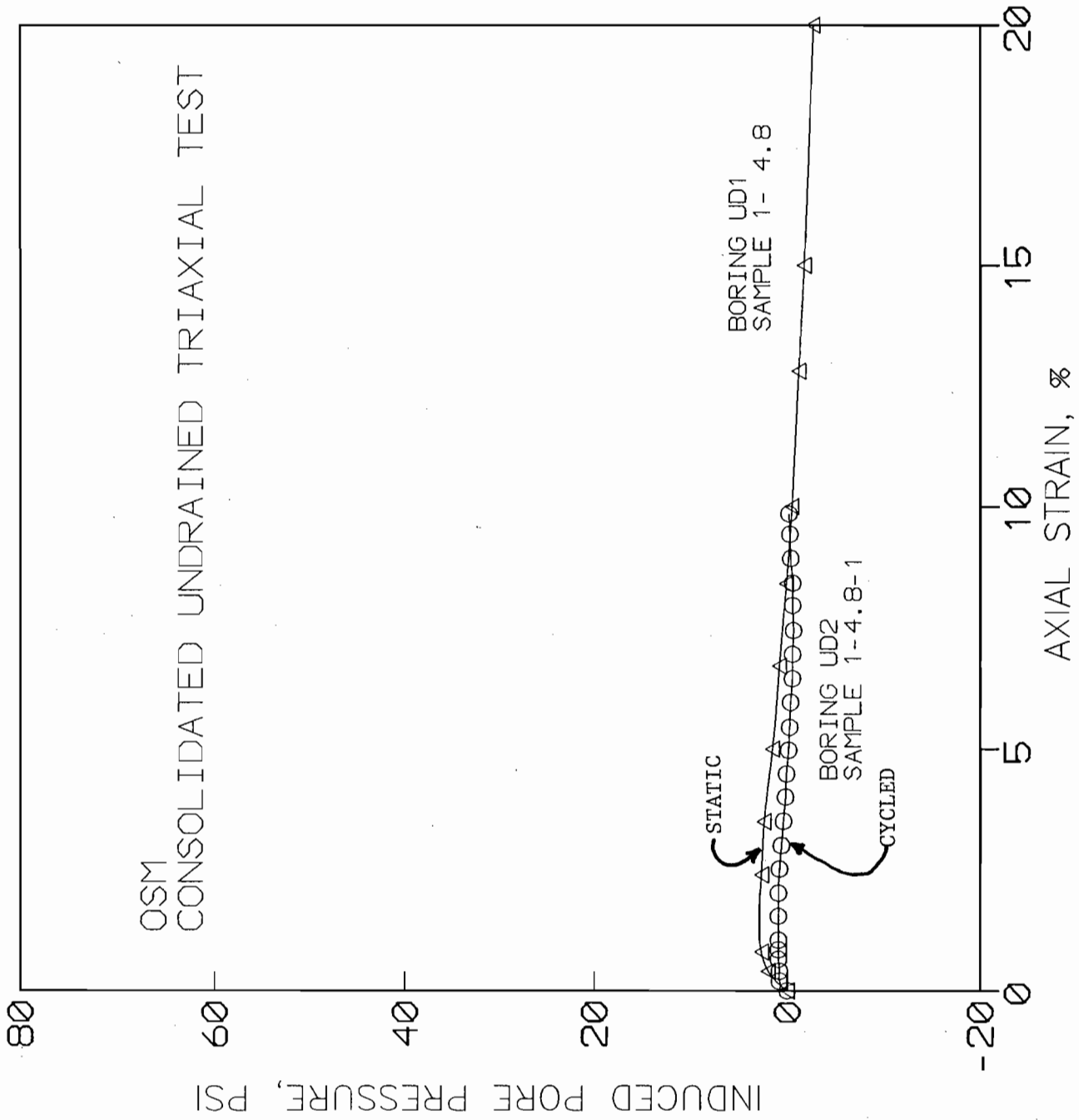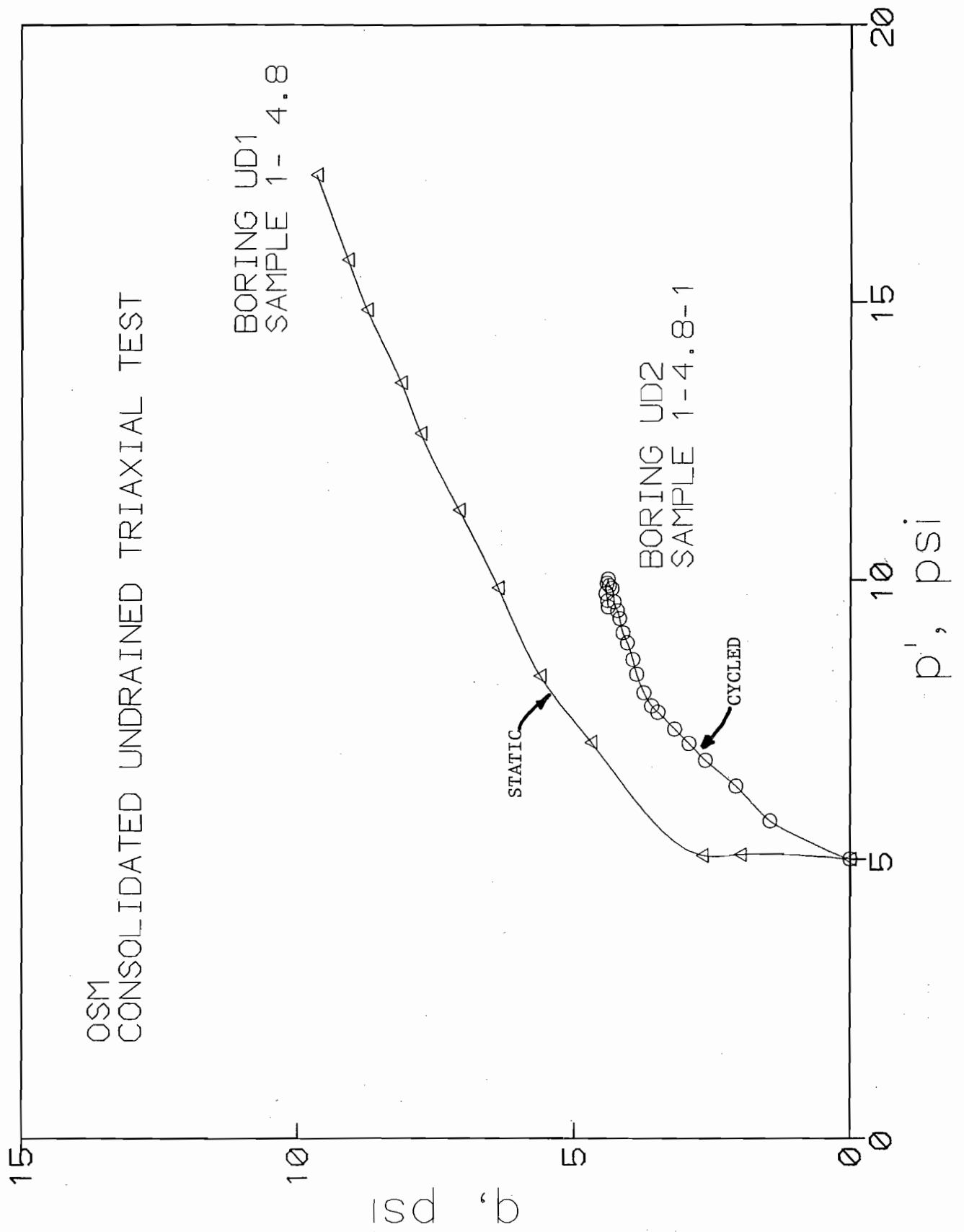
-24-
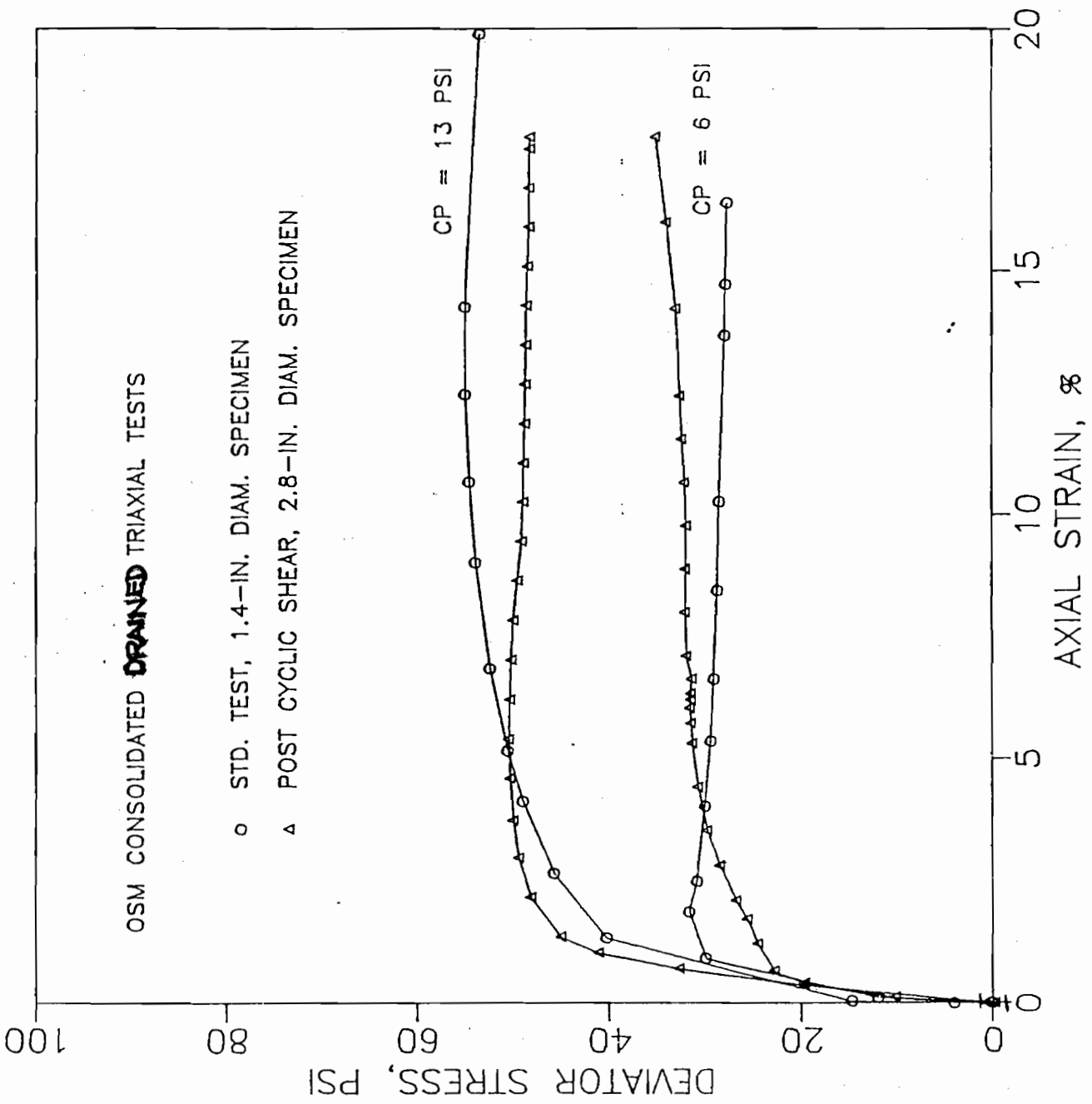
Figure 6. Deviator stress versus axial strain for triaxial tests with and without small amplitude cyclic loading.

United States Department of the Interior

OFFICE OF SURFACE MINING
Reclamation and Enforcement
Eastern Support Center
Ten Parkway Center
Pittsburgh, PA 15220

AUG 2 2 1991

Mr. Chiarito
U.S. Army WES
Attn: EWES-SS-R
3909 Halls Ferry Road
Vicksburg, MS   39180-6199

Subject: Interagency Agreement No. EF68-IA91-13796 Entitled "Field and
           Laboratory Evaluation of Potential Causative Factors of
           Structural Damages in Daylight/McCutchanville, Indiana"

Dear Mr. Chiarito:

Enclosed are six (6) copies of the subject Interagency Agreement.

Please review the Agreement and return five (5) signed copies to my

attention as soon as possible. Do not change or modify the agreement.

If you do not agree with the terms and conditions, please call me at

(412) 937-2837.

Sincerely,

Brian J. Luzik
Contracting Officer

Enclosures

Document No.
Contract No. *FF68·IA91-13796*

EFFECTIVE DATE:

INTERAGENCY AGREEMENT

Between

THE OFFICE OF SURFACE MINING RECLAMATION AND ENFORCEMENT
U.S. DEPARTMENT OF THE INTERIOR

And

CORPS OF ENGINEERS, WATERWAYS EXPERIMENT STATION
U.S. ARMY

FIELD AND LABORATORY EVALUATION OF POTENTIAL CAUSATIVE FACTORS

OF STRUCTURAL DAMAGES IN DAYLIGHT/MCCUTCHANVILLE, INDIANA

Fiscal Year: 1991
Account No.: 1-13-4220-000
Obligated Amount: $106,500.00

INTERAGENCY·AGREEMENT

Between

THE OFFICE OF SURFACE MINING RECLAMATION AND ENFORCEMENT

And

CORPS OF ENGINEERS, WATERWAYS EXPERIMENT STATION

I. OBJECTIVE

At the request of the Indiana Department of Natural Resources (IDNR), the Office of Surface Mining Reclamation and Enforcement (OSM), acting through its Eastern Support Center, has undertaken an investigation of citizens' allegations of structural damages from local surface mine blasting in Daylight and McCutchanville, Vanderburgh County, Indiana. The Ayrshire Mine of the AMAX Coal Company is the focal point of blasting complaints in the study area. The mine began operations in 1973 and progressed from the eastern boundary of the permit to within 3.5 miles east of McCutchanville and 2 miles east of Daylight. To date, several phases of investigation have been completed by the IDNR and OSM. Significant and widespread occurrences of structural damage in the study area have been documented. It has also been established that blasting related ground vibrations and/or airblasts from the Aryshire Mine are discernable to the complainants.

A November 1989 - January 1990 study by the U.S. Bureau of Mines (USBM) involved monitoring of ground vibrations, the structural responses to those vibrations, and potential crack development in building materials during on-going operations at the Ayrshire mine. This study found no clear correlation between blasting and crack formation or extension in the studied structures. The maximum amplitude of recorded ground vibration and the resulting structure vibration were found to be well below the established thresholds for cosmetic damage. However, in-house and interagency reviews of the OSM investigation up to and including the USBM study identified a number of outstanding technical issues. These issues include the following:

1) Is there a potential for collapse of the structure of unsaturated soils or pore-pressure rise in saturated soils in the study area due to ground vibration?

2) Can observed damage be ascribed to fatigue induced by the repetitive exposure of structures to ground vibrations and/or airblasts?

3) Are there ground vibrations at very low frequencies (down to 0.5 Hz.) that are capable of causing structural damage?

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4)   Are there comparable damages in a remote area (unaffected by blasting) with similar geology, soils, and topography?

5)   Do airblasts produce adverse structural response in the study area?

6)   Certain types of structural damages, observed by some investigators, appear to have been caused by lateral forces. If so, what are the relative contributions of blast-induced ground vibrations/airblasts, earthquakes, and wind to this force?

7)   Do alternative mechanisms (inadequate foundations, slope/soil movement) contribute to the observed damages?

8)   To what degree do geology, soil, and topography influence ground wave propagation, site response amplification, and the amplitude, frequency, and duration of waves?

9)   To what extent does blast design (both conventional and cast blasting) alter the effects of blast vibrations in the study area?

II. BACKGROUND

The work to be performed under this is Agreement will be an integral part of an interagency study aimed at resolving the above issues. Other agencies participating in this study are the U.S. Bureau of Mines (USBM) and the U.S. Geological Survey (USGS). The tasks to be performed specifically by the Corps of Engineers, Waterways Experiment Station (WES) are designed to address Issues 1, 2, 3, 5, 6, and 7. Technical support to this Agreement will be provided by the IDNR and OSM.

Authority to enter into this Interagency Agreement is contained in the Surface Mining Control and Reclamation Act of 1977 (P.L. 95-87) and the Economy Act (P.L. 97-258).

III. TERMS OF AGREEMENT

This Agreement may be amended by mutual consent of both parties in writing. The period of performance of this Agreement shall be for one year from date of acceptance. It shall continue in force unless modified by mutual consent or terminated by either party by written notice to the other party at least 30 days prior to the termination date. Due to the nature of field and analysis tasks being undertaken and the required schedule for completion, it is acknowledged that the Agreement will span portions of FY 1991 and FY 1992.

IV. STATEMENT OF WORK

A.   OSM agrees to:

 1.   Provide personnel for the purpose of coordinating site selection and

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other field activities affecting structure analyses; and ground vibration, airblast and structure response monitoring.

2. Obtain all rights of entry and all other government clearances for property access.

3. Provide geophysical and shallow drilling and undisturbed sampling services, through a contractor or government agency, for purposes of collecting soil samples from sites in Daylight, McCutchanville and a "remote" area (unaffected by blasting). Exact sampling procedures and locations and depths will be selected by OSM in consultation with the principal investigator.

4. Provide soil samples to WES for transport to the Waterways Experiment Station in Vicksburg, Mississippi for cyclic load testing.

5. Reimburse WES for personnel, equipment, materials, travel, per diem and other expenses incurred in performing tasks under this IA up to the amount of $167,075.

B. The WES agrees to:

1. Perform testing and modeling services in the field and lab as per the following Scope of Work:

IN-FIELD MONITORING AND STRUCTURAL DYNAMICS ANALYSES

a. Select one structure in the study area for load failure analyses. Select one structure in the study area for monitoring ground vibrations, airblasts and structural response.

b. Conduct engineering analyses on selected structure to (1) estimate vertical wall loads on footings, (2) determine probable extent of foundation settlement from estimated static wall loads and (3) determine differential settlements required to cause yield line cracking in unreinforced basement floor slabs. *GL data avail.*

c. Conduct lateral load analyses for unreinforced basement walls in selected structure as follows: (1) Develop realistic bounding values for lateral earth pressures on basement walls, to include probable values for confined swell pressures in expansive clays; (2) estimate vertical loads on the walls; (3) estimate structural strength of the walls; and (4) estimate onset of cracking in the walls, using values for lateral earth pressures, vertical wall loads and wall strength. *GL data avail.*

d. Monitor free-field and near-structure ground vibrations; airblast distributions on mine-facing side of structure; and structural response during surface mine blasting activity and other sources of cyclic loading. Monitor ground vibrations in the range of 0.5 to 60 Hz. Also conduct a modal test to identify overall and component dynamic properties of structure. Use data to determine

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energy levels of very low frequency vibrations; and interrelationships between exterior dynamic loadings and structural response.

e. Perform multi-degree-of-freedom and fatigue analyses using a structural model (one-story and two-story) based on information obtained under Task 1.d. Estimate minimum stress levels that could cause cracking and/or other damage based on various scenarios pertaining to dynamic loading parameters, material prestrain levels and fatigue. Determine whether relationship exists between common crack patterns in the study area and cyclic loading.

LABORATORY SOIL TESTING

f. Test soil samples for consolidation under induced cyclic loading as follows: Apply cyclic loading tests to 12 samples obtained from Daylight, McCutchanville and the remote area by OSM. Between 12 and 24 tests shall be conducted using a Drnevich resonant column loading device. Each tested sample shall be drained and subjected to 30,000 cyclic loadings in a frequency range of 4 to 20 Hz. All 12 samples shall initially be tested at two separate shear strain levels, the largest of which shall be based on the highest observed peak particle velocity measured in the study area. Further testing at 1/10 the original shear strain level shall be performed only if consolidation is detected in the initial results. G_L

g. If consolidation occurs in testing under Task 1.f., evaluate potential damaging effects of soil consolidation beneath structural foundations. The evaluation shall be based on available site-specific soil data as well as the test results. G_1

h. Conduct two pilot undrained cyclic triaxial tests and two companion static undrained triaxial tests to failure on saturated specimens from the study area. Use a vertical strain level equal to twice the maximum shear strain level used under Task 1.f. Assess whether significant strength degradation occurs as a result of low level cyclic loading. If significant strength degradation is determined, recommend further testing not funded under this IA. G

i. If significant strength degradation is determined under Task 1.h., develop a chart showing effect of degradation on slope stability. G_1

2. Attend meetings with other interagency team members from USGS, USBM, OSM and IDNR. Present preliminary findings, recommend project modifications where appropriate and identify support/coordination* requirements for remaining activities. The exact time and place of the meetings shall be agreed upon by all project participants. SL
                                                                            G_L

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. 3. Following completion of all field work and analysis, meet with the interagency team to develop an interagency draft report on the results of the investigation.

4. Perform services according to the following schedule:

a. Mobilize necessary personnel and equipment to the study area for field operations upon selection of study sites.

b. Attend an interagency team meeting for evaluation of project progress and consideration of possible project modifications within eighty (80) calendar days of this Agreement's effective date.

c. Complete field monitoring and all other data-gathering activities within one hundred and five (105) calendar days of this Agreement's effective date.

d. Complete all data analyses within one hundred and eighty (180) calendar days of this Agreement's effective date.

e. Attend (with written WES draft report in hand) an interagency team meeting to develop composite draft final report following performance of 4.d, but within two hundred and twenty (220) calendar days of this Agreement's effective date.

f. Complete the review of the interagency team composite draft final report following the performance of 4.e, but within two hundred and thirty (230) calendar days of this Agreement's effective date.

V. KEY OFFICIALS

The project officers shall be:

Peter Michael (COTR)
Office of Surface Mining
Reclamation and Enforcement
Eastern Support Center
Ten Parkway Center
Pittsburgh, PA 15220
412-937-2867

Vincent P. Chiarito
U.S. Army Engineer WES
Attn: CEWES-SS-R
3909 Halls Ferry Road
Vicksburg, MS 39180-6199
601-634-2714

The principal investigators for WES will be Vincent Chiarito and Dr. Paul Hadala.

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VI. FUNDING

OSM will fund this Agreement by obligations under its Regulatory and Technology Activity using a combination of FY 1991 and FY 1992 funds. FY 1991 funds in the amount of $106,500 will be obligated with the execution of this agreement and will remain available to the WES until expended. The remaining $60,575 will be obligated from FY 1992 funds on or after October 1, 1991.

VII. PAYMENT

Monthly payment will be made by OSM upon receipt of a properly executed Standard Form 1081 and the required monthly report. The form shall be submitted to:

OFFICE OF SURFACE MINING RECLAMATION AND ENFORCEMENT
Eastern Support Center
Ten Parkway Center
Pittsburgh, PA 15220
ATTN: Management Services Branch

VIII. DELIVERABLES

The following items are deliverables under this Agreement:

a. Three copies of monthly letter reports outlining monthly activities, problems encountered, and budget status are to be submitted to the OSM project officer by the 7th of the following month.

b. One hard copy and 5 1/4 inch diskettes of the agency's draft report for incorporation into the interagency team report. The report diskettes are to be on software compatible with "Word Perfect" version 5.0.

c. One set of 5 1/4 inch diskettes containing all databases of field data and analyses. The applicable software for operation must be identified for each database.

d. The following deliverables are to be completed prior to OSM's acceptance of work performed under this Agreement:

    i.   Submission and acceptance of the interagency team's composite report;

    ii. Submission of a copy of all field data gathered during the investigation including photography, data logs/records, and laboratory test results; and

    iii. Participation of the principal investigator in an

<div align="center">7</div>

interagency briefing of OSM management staff following
submission of the final team report to OSM.

IX. EVALUATIONS

OSM reserves the right to make programmatic evaluations of the work
carried out by the WES under this Agreement, including field or
laboratory site visits. Any such visits will be made with the prior
knowledge of the WES project officer. Appropriate and mutually
agreeable overview procedures will be established by the WES and OSM
project officers to adjudicate review results in the case of WES-OSM
disagreement.

X. PUBLIC INFORMATION

All information obtained under the terms of this Agreement is public
property.

Copies of all scientific publications of the results of research under
this Agreement and any press releases prepared by WES regarding this
Agreement and/or any subagreements will be forwarded for review to the
cognizant Project Officer prior to public release or presentation. In
all such cases, credit for joint support to WES and OSM shall be
acknowledged in all printable material. In the case of failure to
agree as to the interpretation of results, either party may publish
data after due notice and submission of the proposed manuscript to the
other. In such instances, the party publishing the data will duly
credit the cooperation of the other party, but will assume
responsibility for any statements on which there is a difference of
opinion. To prevent disclosure of information requested to be kept
confidential by third parties and prohibited from disclosure by Federal
law and to protect potential patent and invention rights, Project
Officers shall seek advice of their respective General Counsel's office
as appropriate. Provisions of this Agreement cannot supersede public
disclosure requirements of the Freedom of Information Act.

XI. MISCELLANEOUS

During the performance of this Agreement, the participants agree to
abide by the terms of Executive Order 11246 on non-discrimination and
will not discriminate against ant person because of race, color,
religion, sex, or national origin. The participants will take
affirmative action to ensure that applicants are employed without
regard to their race, color, religion, sex, or national origin.

No member or delegate to Congress, or resident Commissioner, shall be
admitted to any share or part of this Agreement, or to any benefit, that
may arise therefrom, but this provision shall not be construed to
extend to this agreement it made with a corporation or its general
welfare.

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Signed this _____ day of _____, 1991.

UNITED STATES DEPARTMENT OF THE INTERIOR
OFFICE OF SURFACE MINING RECLAMATION AND ENFORCEMENT

By:_____

Title:_____

Accepted this _____ day of _____, 1991.

UNITED STATES ARMY, CORPS OF ENGINEERS
WATERWAYS EXPERIMENT STATION

By:_____

Title_____

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2L continued to angle & 27 for angle
delivt to hand run in 33°C at 9/24

BORING LOG
FIELD DATA

Project: OSM - Undisturbed Sampling   Site _____   Date 5/19/92

Location: Zimmerman Property   Job No. _____

Drill Rig: _____   Inspector: Vispi   Operator: _____   Surface El _____   Boring No. 1   UD-1

SAMPLE NUMBER	DATE TAKEN	STRATUM		DRIVE		SAMPLE		TYPE OF SAMPLER	Hyd. Press	Cont.	CLASSIFICATION AND REMARKS
		FROM	TO	FROM	TO	FROM	TO				
				0.0	4.0			6" Flight Auger			Advance Boring
1	5/19			4.0	5.0	4.0	5.0	5" Horsley Sampler	500	CT	Chucking Vise slipped
1A	5/19			5.0	5.38	5.0	5.18	Fixed Piston Sampler	200	J	Drove 1.38' Bottom of spl. not retrieved Gap ~0.21'- Material is so "sticky" it would not break @ bottom of tube. 1.18' of spl. recovered. Quality of spl. is questionable. Clay & silt with silt & silt nodules, moist brown & gray.
	5/20			4.0	5.6			6" Flight Auger			Cleanout & advance boring Water in bottom of boring

BORING LOG
FIELD DATA

Project OSM - Undisturbed Sampling Site _____ Date 5/20/92

Location Zimmerman Property Job No. _____

Drill Rig _____ Inspector Visp. Operator _____ Surface El _____ Boring No. 1

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Hyd. Press.	Cont.	CLASSIFICATION AND REMARKS
											0.00' Gap
2	5/20			5.6	6.1			5' Hvorslev	500		2.06' Drive (max. allowable
				6.1	6.6			Fixed Piston	700		hyd. stroke) 1.6' Spl.
				6.6	7.1	5.7	7.1	Sampler	750	CT	recovered. Hole measured
2A	5/20			7.1	7.6	7.1	7.2		800	J	to 7.2'. Material is very
				7.6	7.66				800		sticky, not breaking off
											@ bottom of tube. Clay &
											silt w/ silt and weathered
											shale nodules moist brown
											and gray.
				5.6	22.5			6" Flight Auger			Advance Boring. Hole
								Auger			squeezing in @ 15.0' (soft
											wet silt) several attempts
											failed. Could not get sampler
											past 15'.
											End of Boring

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BORING LOG
FIELD DATA

Project __OSM – Undisturbed Sampling__   Site _____   Date __5/20/92__
Location __Zimmerman Property – 5' W-SW of Boring 1__   Job No. _____
Drill Rig _____   Inspector _____   Operator __Vispi__   Surface El _____   Boring No. __1__

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Hyd. Press	Cov. %	CLASSIFICATION AND REMARKS
				0.0	4.1			6" Flight Auger			Advance Boring
1	5/20			4.1	4.6			5" Hvorslev	300		2.55' Drive – 0.01' Gap
1A	5/20			4.6	5.1	4.15	5.15	Fixed Piston Sampler	350	CT	Bottom 0.3' of Spl. not recovered. (material not recovered)
				5.1	5.6	5.15	5.20		400	J	
2	5/20			5.6	6.1	5.20	6.25		4.50	CT	breaking @ bottom of tube
2A	5/20			6.1	6.68	6.25	6.30		600	J	Clay & Silt w/ silt & weathered shale nodules moist, brown & gray.
				4.1	12.0			6" Flight Auger			Clean out & Advance Boring.

BORING LOG
FIELD DATA

Project CSM - Undisturbed Sampling Site _____ Date 5/20/92

Location Zimmerman Property - 5' N-SW of Boring 1 Job No. _____

Drill Rig _____ Inspector Vispi Operator _____ Surface El _____ Boring No. 2

SAMPLE NUMBER	DATE TAKEN	STRATUM		DRIVE		SAMPLE		TYPE OF SAMPLER	Hyd. Press.	Con.T.	CLASSIFICATION AND REMARKS
		FROM	TO	FROM	TO	FROM	TO				
3	5/20			12.0	12.5			5' Hvorslev	800		2.5E' Push - 0.00' Gap
3A	5/20			12.5	13.0	12.05	13.15	Fixed Piston	850	C.T	Clay & silt w/silt E.
4	5/20			13.0	13.5	13.15	13.20	Sampler	900	J	Weathered shale fragments
4A	5/20			13.5	14.0	13.20	14.40		950	C.T	moist, brown & gray.
				14.0	14.58	14.40	14.5		1000	J	
											End of Boring
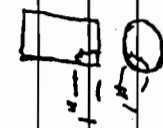
WES FORM 819
JAN 74   EDITION OF NOV. 1971 MAY BE USED

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BORING LOG
FIELD DATA

Project __OSM - Undisturbed Sampling__   Site _____   Date __5/20/91__

Location __Zimmerman Property - 5'-W-SW of Boring 2__   Job No. _____

Drill Rig _____   Inspector __Vispi__   Operator _____   Surface El _____   Boring No. __3__

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Hyd. press.	Con'f.	CLASSIFICATION AND REMARKS
1				0.0	4.0			6" Flight Auger			Advance Boring
1A	5/20			4.0	4.5	4.05	4.5	5" Hvorslev	400	CT	2.5' Drive - Bottom 0.12'
	5/20			4.5	5.0	5.15	5.20	Fixed Piston	800	J	of sample not recovered.
2	5/20			5.0	5.5	5.20	6.35	Sampler	850	CT	0.015 Gap. Clay & 5.17 w/
2A	5/20			5.5	6.0	6.35	6.45		900	J	silt & weathered shale
				6.0	6.5?				1000		nodules, moist, brown & gray
				4.0	12.2			6" Flight Auger			Clean out & advance Boring
											Water in bottom of boring.

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BORING LOG
FIELD DATA

Project **Riney Property - 2'N & 1'W of Boring 8** Site ____ Date **5/22/92**

Location ____ Job No. ____

Drill Rig ____ Inspector **Visqi** Operator ____ Surface El ____ Boring No. **10**

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Hyd. press.	conf. cont.	CLASSIFICATION AND REMARKS
1	5/22			0.0	1.5			6" Flight Auger			Advance Boring
								Auger			
1A	5/22			1.5	2.0	1.55	2.65	5" Hvorslev	400	CT	2.50 Drive, 0.005' Gap
2	5/22			2.0	2.5	2.65	2.75	Fixed	500	J	Silt & Clay, sl. moist
2A	5/22			2.5	3.0	2.75	3.83	Piston	600	CT	Mod. firm, roots to .75
	5/22			3.0	3.5	3.83	4.00	Sampler	900	J	brown w/ tan spots,
				3.5	4.0				800		somewhat blocky
											End of Boring

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Sheet __1__ of __1__ Sheets

BORING LOG
FIELD DATA

Project: Riney Property; 4' N of Boring B
Location: Riney Property   Site: Boring B   Date: 5/22/92
Job No. _____
Drill Rig _____   Inspector _____   Operator: Vispi   Surface El _____   Boring No. 9

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Hyd Pres	Cont.	CLASSIFICATION AND REMARKS
				0.0	1.5			6" Flight Auger			Advance Boring
1	5/22			1.5	2.0	1.52	2.55	5' Hvorsley Fixed	300	CT	2.53' Drive 0.005' Gap
1A	5/22			2.0	2.5	2.55	2.75	Fixed	400	J	Silt & Clay, sl. moist mod.
2	5/22			2.5	3.0	2.75	3.97	Piston	600	CT	firm trace of shale nodules
2A	5/22			3.0	3.5	3.97	4.03	Sampler	250	J	rootlets brown w/ Lt. Tan to gray streaks, somewhat blocky
				3.5	4.03				900		1"x1" plug out of the bottom side of spl. #1 & top side of spl #2
											End of Boring

Sheet 1 of 1 Sheets

BORING LOG
FIELD DATA

Project __Riney Property__ Site _____ Date __5/21/92__

Location __5' N of Boring 7__ Job No. _____

Drill Rig _____ Operator _____ Boring No. __6__

Inspector _____ Surface El _____

SAMPLE NUMBER	DATE TAKEN	STRATUM		DRIVE		SAMPLE		TYPE OF SAMPLER	Hyd.* Pres. (g")	X	CLASSIFICATION AND REMARKS
		FROM	TO	FROM	TO	FROM	TO				
				0.0	4.0			6" Flight Auger			Advance Boring
1	5/21			4.0	4.5	4.05	4.15	5" Hvorslev	400	CT	2.44' Drive – 0.01' Gap
1A	5/21			4.5	5.0	5.15	5.25	Fixed Piston	500	J	Clay & Silt (mostly silt)
2	5/21			5.0	5.5	5.25	6.35	Sampler	600	CT	w/ weathered shale nodules
2A	5/21			5.5	6.0	6.35	6.44		700	J	Moderately firm, sl. moist,
				6.0	6.44				800		brown w/ gray streaks.
											Sandstone nodules at
											bottom of spl. #2
											End of Boring

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BORING LOG
FIELD DATA

Project __Riney Property__ Site _____ Date __5/21/92__

Location __Riney Property__ Job No. _____

Drill Rig _____ Inspector __Vispi__ Operator _____ Surface El _____ Boring No. __7__

SAMPLE NUMBER	DATE TAKEN	STRATUM		DRIVE		SAMPLE		TYPE OF SAMPLER	blows per hole	Cor %	CLASSIFICATION AND REMARKS
		FROM	TO	FROM	TO	FROM	TO				
								6" Flight Auger			Advance Boring
1	5/21			4.0	4.5	4.0	5.0	5" Hvorsley	450	CT	2.38' Drive, 0.00' Gap
1A	5/21			4.5	5.0	5.0	5.1	Fixed	550	J	Left bottom 0.1' in hole
2	5/21			5.0	5.5	5.1	6.15	Piston	600	CT	Clay & silt (mostly silt)
2A	5/21			5.5	6.0	6.15	6.25	Sampler	700	J	w/ weathered shale nodules
				6.0	6.38				800		moderately firm, brown
											w/ gray streaks, sl. moist
				4.0	9.0			6" Flight Auger			Clean out & advance boring
											Hit weathered shale or iron
											nodules (large size)
											between 6.5' & 8.5'. Hit a
											siltstone at 8.5'. Drove a
											Split Spoon to refusal @ 9.0'.
											End of Boring

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BORING LOG
FIELD DATA

Project __Kansas Rd. (OSM-5)__
Location __10' W of Boring 5__   Site _____   Date __5/21/92.__
Drill Rig __V-150__   Inspector _____   Operator _____   Surface El _____
Job No. _____   Boring No. __6__

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Hyd. Press	% Rec.	CLASSIFICATION AND REMARKS
3				4.0	9.1			6" Flight Auger			Clean out & Advance Boring
3A	5/21			9.1	9.6	9.15	9.95	5" Hvorslev	600	CT	1.83' Drive - Used max. push capacity of rig. 0.00' Gap
4	5/21			9.6	10.1	9.95	10.00	Fixed Piston	1200	J	
4A	5/21			10.1	10.6	10.00	10.85	BS Sampler	1400	CT	Clay, £ 5.17 w/silt &
	5/21			10.6	10.93	10.85	10.93		1500	J	Weathered shale nodules; moist, firm, brown & gray

Top of Spl #3 wet because of water in boring. Frequency of nodules appears to be greater in Spl. #3.

Static water level in Boring is 4.0'

End of Boring

Sheet __2__ of __2__ Sheets

BORING LOG
FIELD DATA

Project: __Kansas Rd. (CSM-5)__
Site: __10'W of Boring #5__ Date __5/21/92__
Location: __Kansas Rd. (CSM-5)__ Job No. _____
Drill Rig _____ Inspector __V15pi__ Operator _____ Surface El _____ Boring No. __6__

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Hyd. press.	Con't.	CLASSIFICATION AND REMARKS
1	5/21			0.0	4.0			6" Flight Auger			Advance Boring
											Water in bottom of boring
1A	5/21			4.0	4.5	4.2	5.25	5" Fixed	350	CT	2.61' Drive, 0.00' Gap
				4.5	5.0	5.25	5.35	Piston	550	J	Clay & Silt w/ silt E.
2	5/21			5.0	5.5	5.35	6.50	Sampler	650	CT	Weathered shale nodules,
2A	5/21			5.5	6.0	6.50	6.64		700	J	very moist tan w/ some
				6.0	6.64				850		gray, sl. soft.
											Spl. cracked @ 4.2'-4.04'
											4.2' discarded. Outside of
											Spl. muddy & wet from water
											in boring - wiped as good
											as possible. Considerable
											amount of shale nodules.
											Outside surface of spl. is
											rough because of these
											nodules.

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BORING LOG
FIELD DATA

Project __Kansas Rd. (COSM-5)__
Location __Kansas Rd. (COSM-5)__ Site __5'W of Boring 4__
Drill Rig _____ Inspector __Vispi__ Operator _____ Surface El _____

Date __5/21/92__
Job No. _____
Boring No. __5__

SAMPLE NUMBER	DATE TAKEN	STRATUM		DRIVE		SAMPLE		TYPE OF SAMPLER	CLASSIFICATION AND REMARKS
		FROM	TO	FROM	TO	FROM	TO		
									Bottom 0.17' of sample
									mixed w/mud - discarded
									Broke top 0.15' of spl.
									during handling - discarded.
									End of Boring

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BORING LOG
FIELD DATA

Project: Kansas Rd. (OSM-5) 5'W of Boring 4
Location: _____ Site _____
Drill-Rig: _____ Inspector: Vispi Operator: _____ Surface El: _____
Date: 5/21/92
Job No.: _____
Boring No.: 5

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Hyd press con't	CLASSIFICATION AND REMARKS
										Drilled this boring because
										Boring 4 had water in it
										at a depth of 40'. Don't
										know if this is actual
										ground water or water
										from a heavy rainstorm
										that took place late in
										the afternoon on 5/20/92.
				0.0	9.0			6"Flight Auger		Advance Boring
										Water in bottom of boring,
1A	5/21			9.0	9.5	9.15	10.0	5"Hvorslev	550 CT	2.27'Dr.-Reached max.
1A	5/21			9.5	10.0		10.05	Fixed	1000 J	push capacity of rig.
2A	5/21			10.0	10.5	10.05	10.95	Piston	1200 CT	0.005' Gap. Clay E 5.14
2A	5/21			10.5	11.0	10.95	11.10	Sampler	1400 J	w/ weathered shale nodules
				11.0	11.27				1500	moist firm brown-gray

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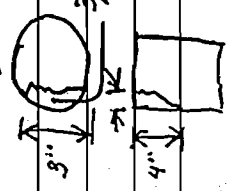
BORING LOG
FIELD DATA

Project _____ Site _____ Date 5/20/92

Location _Kansas Rd. (OSM-5) ≈ 12' N. of Road E_ Job No. _____

Drill Rig _____ Inspector _Vispi_ Operator _____ Surface El _____ Boring No. _4_

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Hyd. press.	cont.	CLASSIFICATION AND REMARKS
								6" Flight Auger			Advance Boring
1	5/20			0.0	4.2						
1A	5/20			4.2	4.7	4.2	5.3	5" Horsley	500	CT	2.46' Drive 0.005' Gap
				4.7	5.2	5.3	5.4	Fixed Piston	650	J	Clay ε Silt w/ silt nodules
2	5/20			5.2	5.7	5.4	6.6	Sampler	800	CT	moist, brown ε gray.
2A	5/20			5.7	6.2	6.6	6.66		900	J	
				6.2	6.66				1100		
											Thunder and Lightening
											Storms hit right after the
											above sample was pushed.
											(@1600 hrs). Move rig off
											hole (because rig was
											partially in roadway) because
											of safety considerations
	5/21	Water in bottom of Boring									
		End of Boring									

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BORING LOG
FIELD DATA

Project OSM - Undisturbed Sampling   Site _____   Date 5/20/91
Location Zimmerman Property - 5'-WSW of Boring 2   Job No. _____
Drill Rig _____   Inspector Visp.   Operator _____   Boring No. 3
Surface El _____

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Hyd. press (Ccn)	Cor %	CLASSIFICATION AND REMARKS
3	5/20			12.2	12.7	12.25	12.7	5' Hverslev	500	CT	2.50" Drive - 0.00' Gap
3A	5/20			12.7	13.2	13.45	13.50	Fixed Piston	800	J	Clay & Silt w/ silt &
4	5/20			13.2	13.7	13.50	14.10	Sampler	1000	CT	weathered shale nodules
4A	5/20			13.7	14.2	14.70	14.78		1100	J	moist, brown & gray
				14.2	14.78				1200		Note - Spl #3 - wedge broke
											out of top

End of Boring

Note: A water cistern is located approx. 45' south of the three undisturbed sample borings on the Zimmerman Property

Sheet 2 of 2 Sheets

WES FORM 819 JAN 74   EDITION OF NOV 1971 MAY BE USED

TEST 13 CH 19 RATE = 488.3 CAL = 1 AVG = -0.008398 VAR = 0.000859 FILE: c:\ateam\newateam\hm051500.fft

FFT TEST 13 CH 19 OVERLAY = 0%   NO HANNING

C64
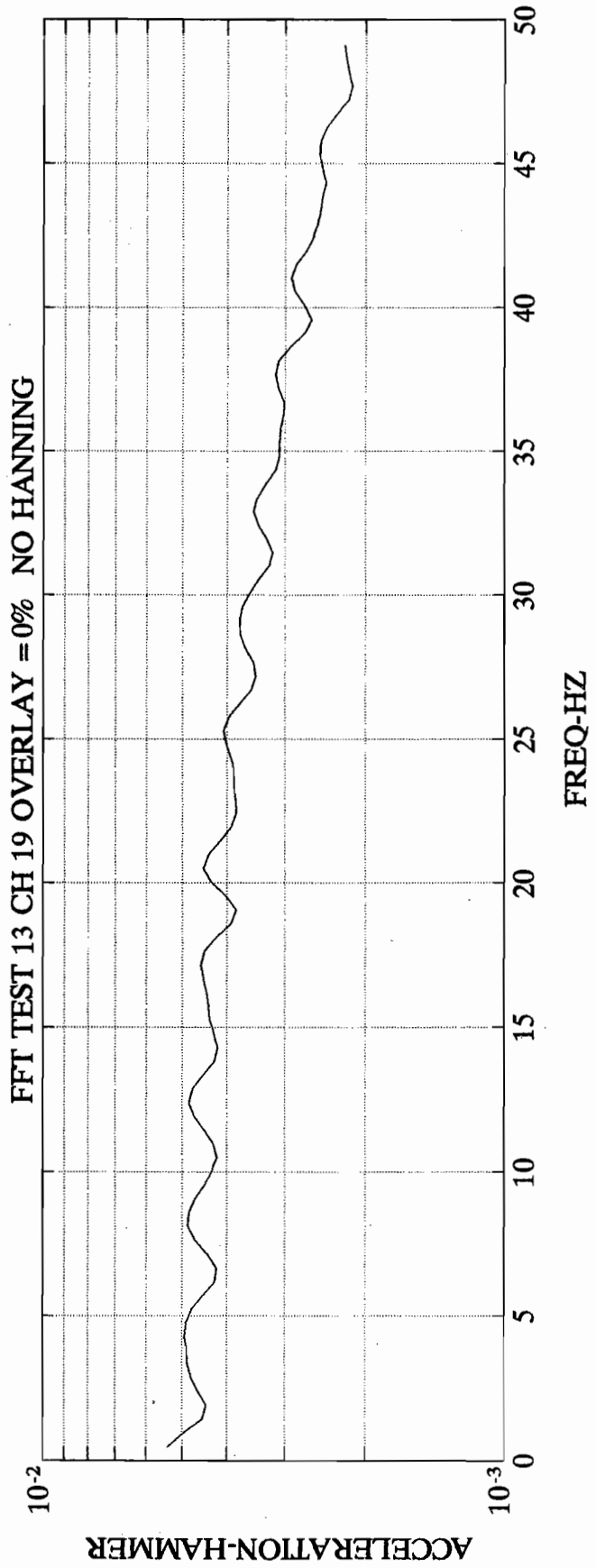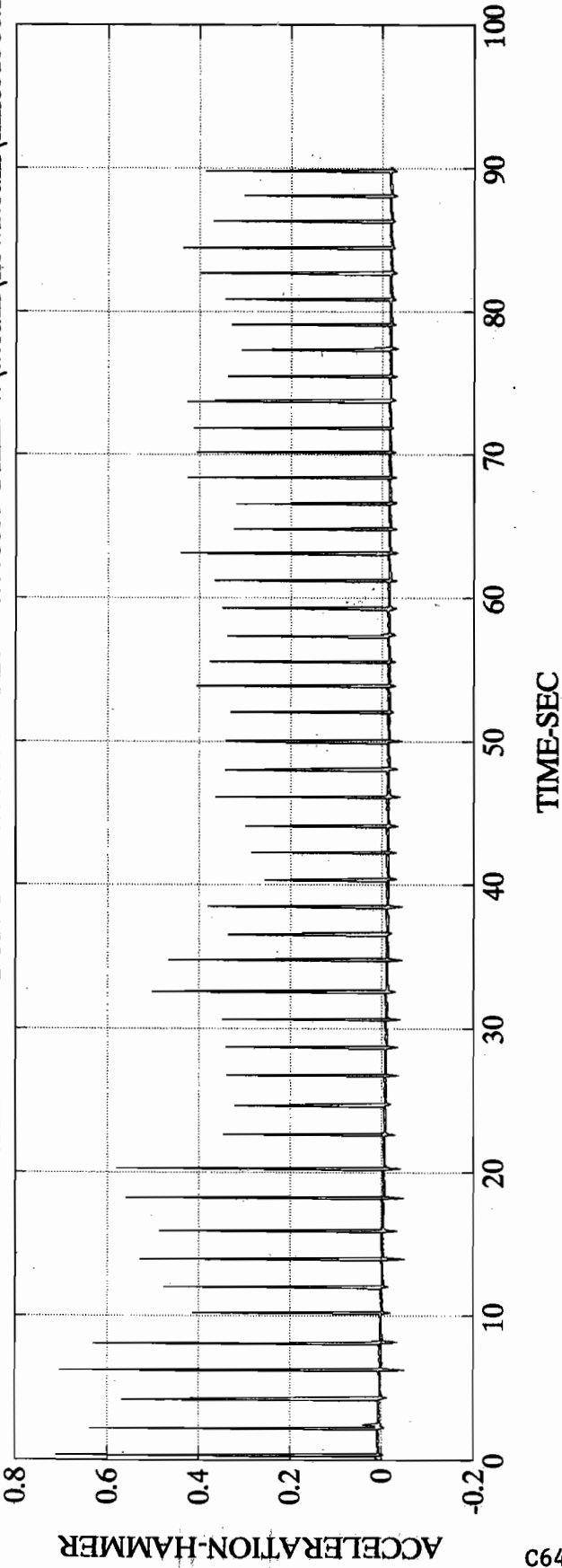
TEST 13 CH 18 RATE = 488.3 CAL = -0.0009579 AVG = 0.0001983 VAR = 5.619e-09 FILE: c:\ateam\newateam\hm0515
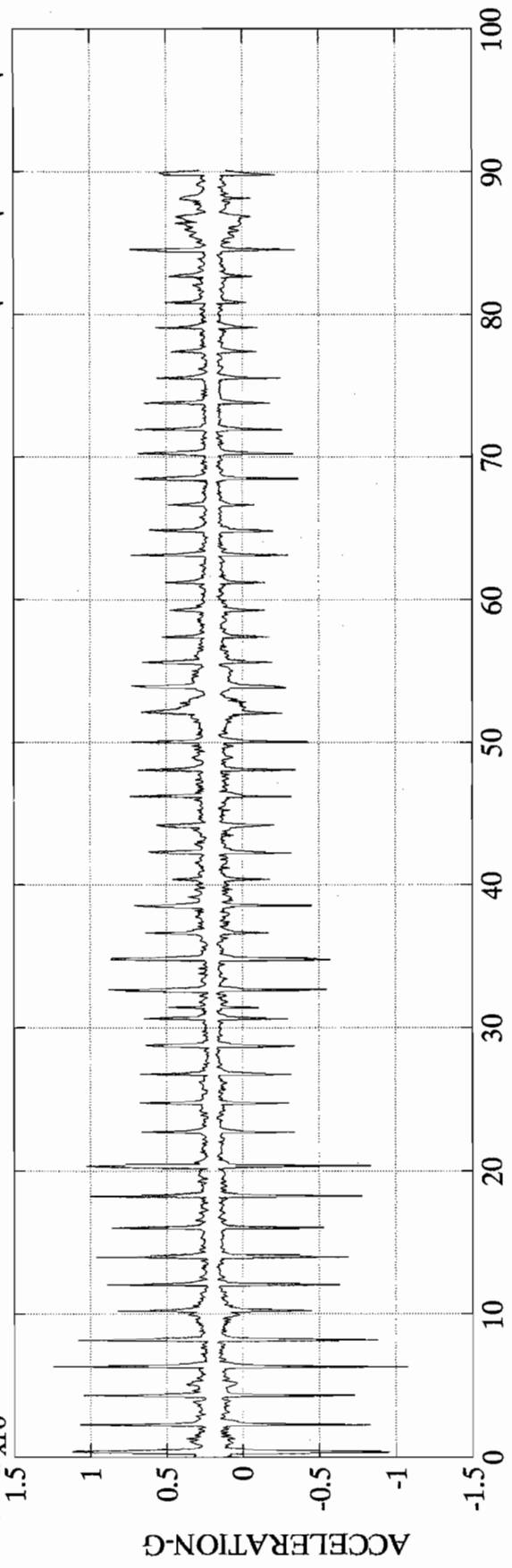
FFT TEST 13 CH 18 OVERLAY =0%   NO HANNING

C63
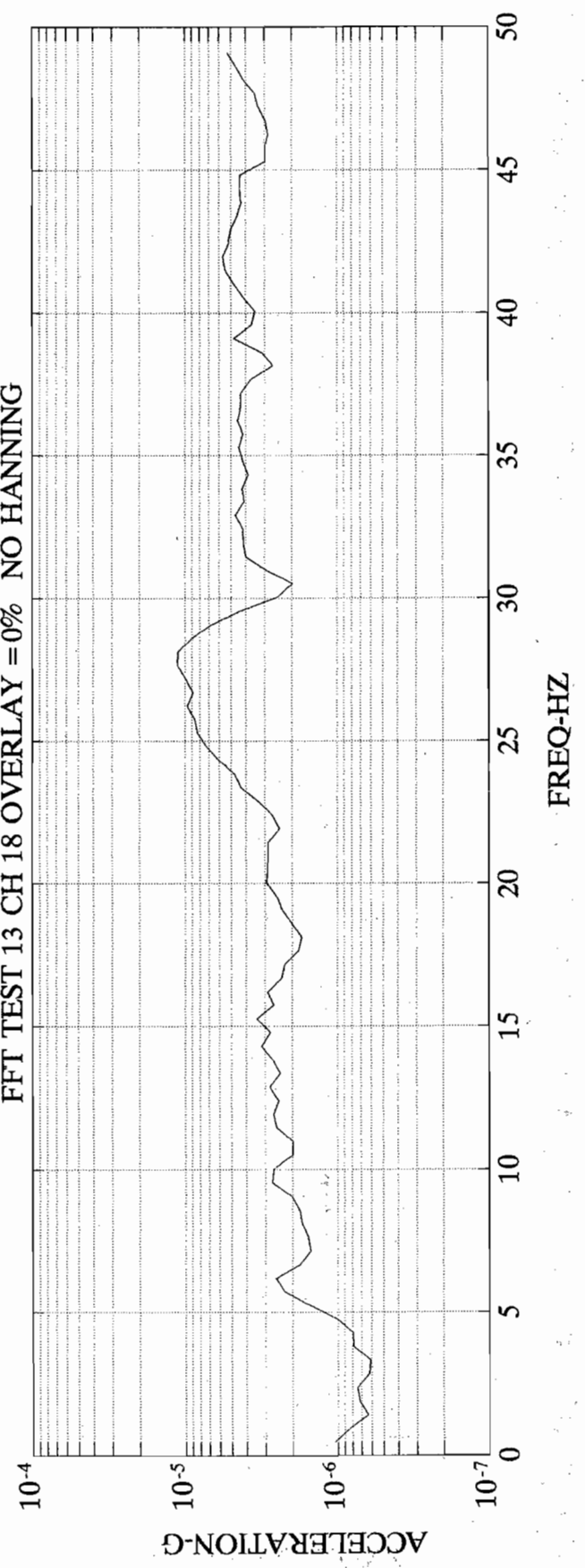
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ACCELERATION-G
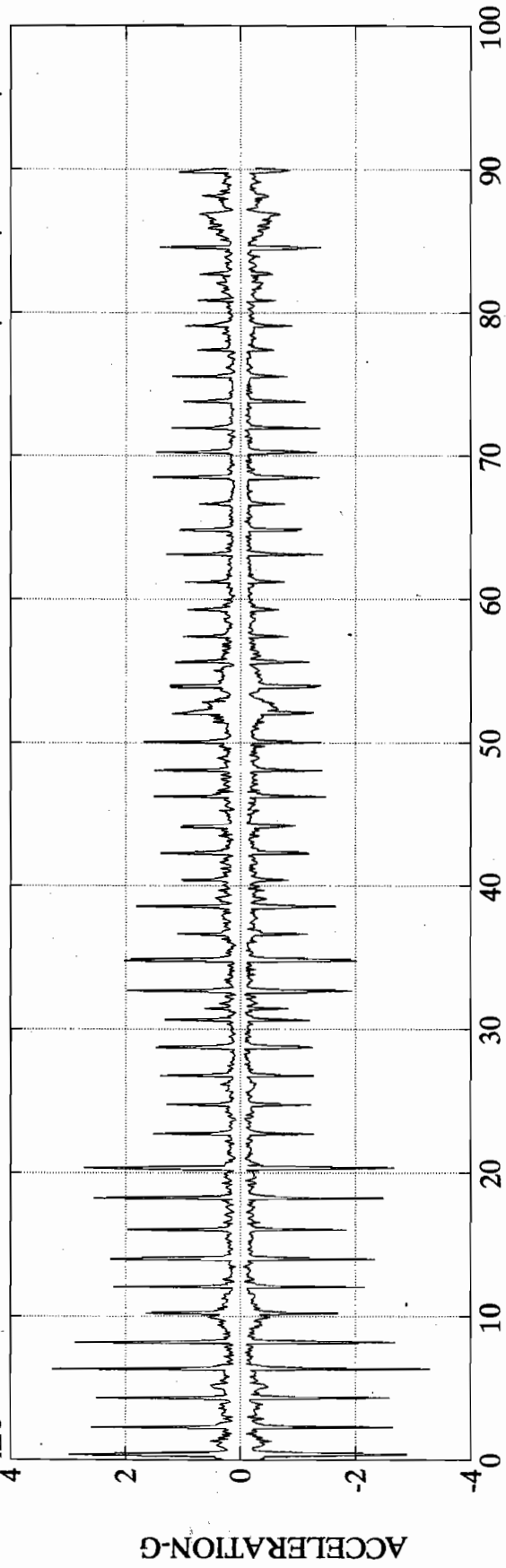
TIME-SEC

FFT TEST 13 CH 17 OVERLAY = 0% NO HANNING

ACCELERATION-G

FREQ-HZ

C62
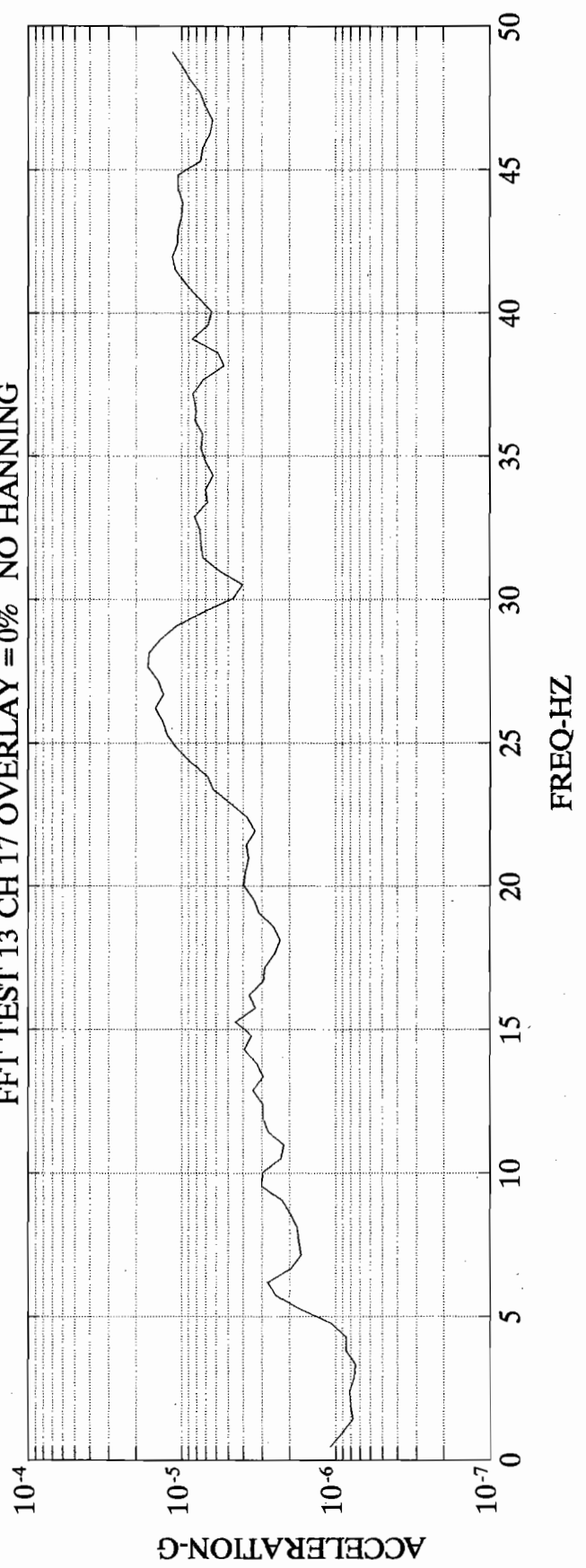
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ACCELERATION-G

TIME-SEC
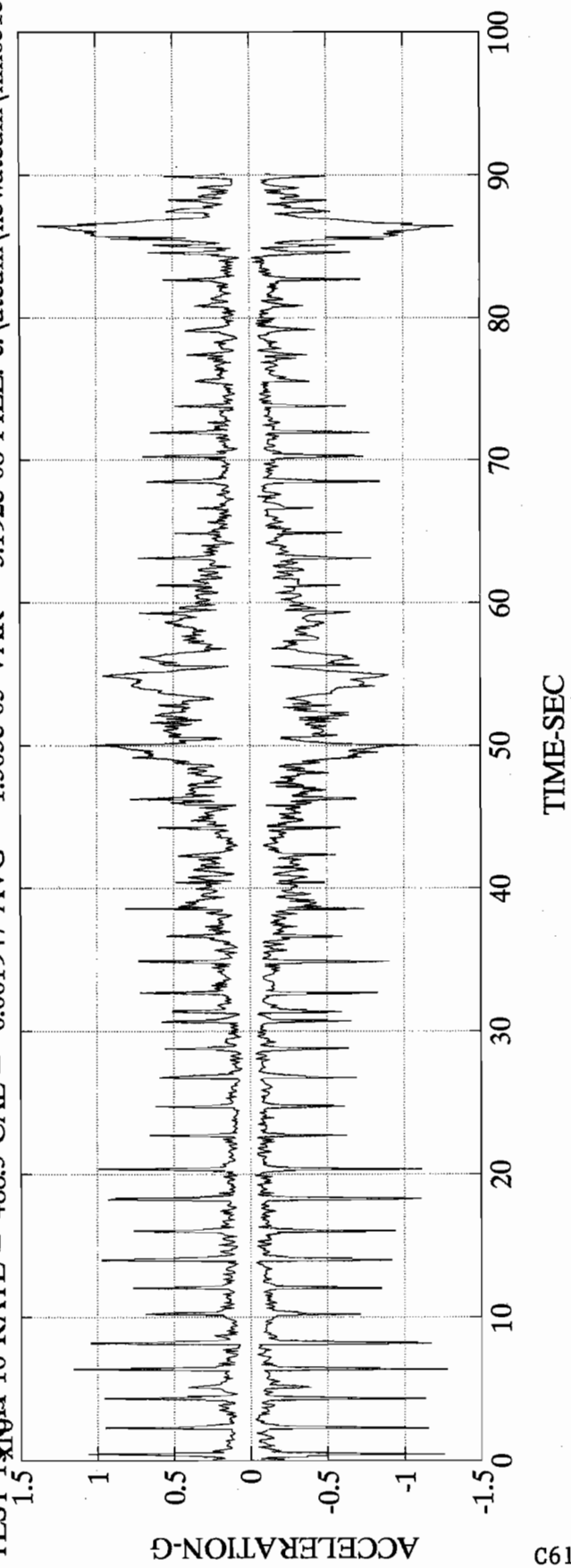
FFT TEST 13 CH 16 OVERLAY =0%   NO HANNING

ACCELERATION-G

FREQ-HZ

C61
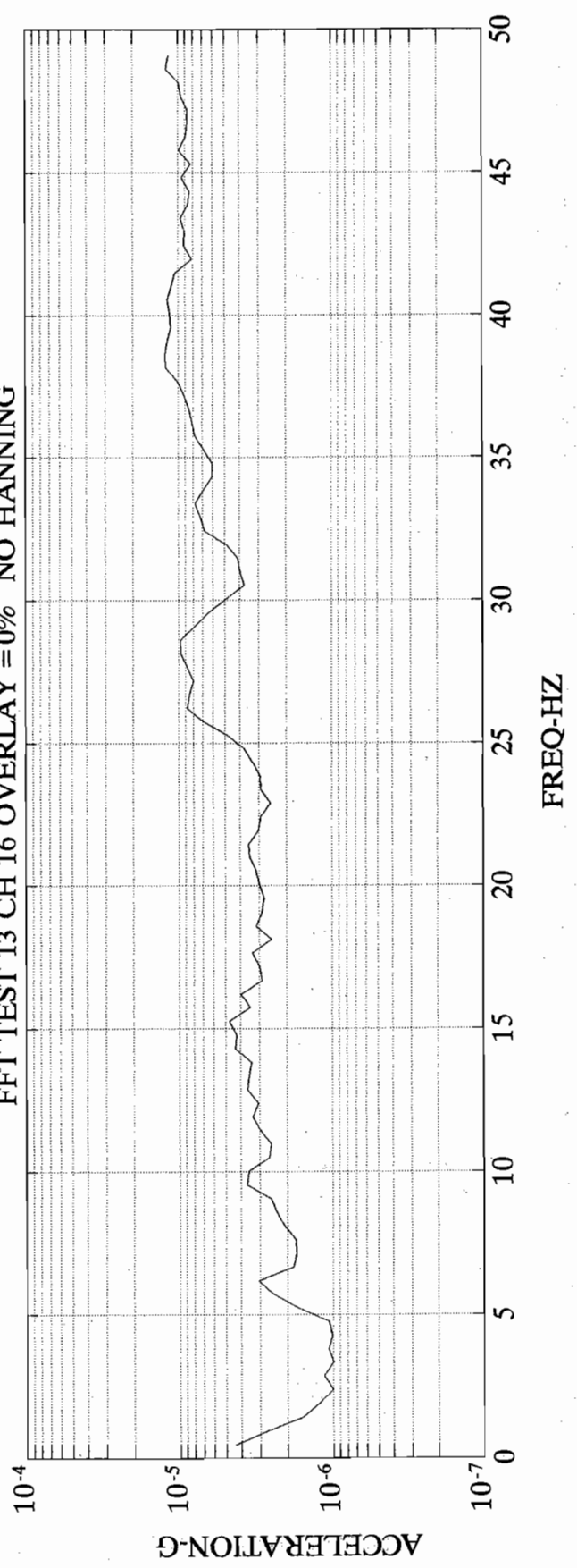
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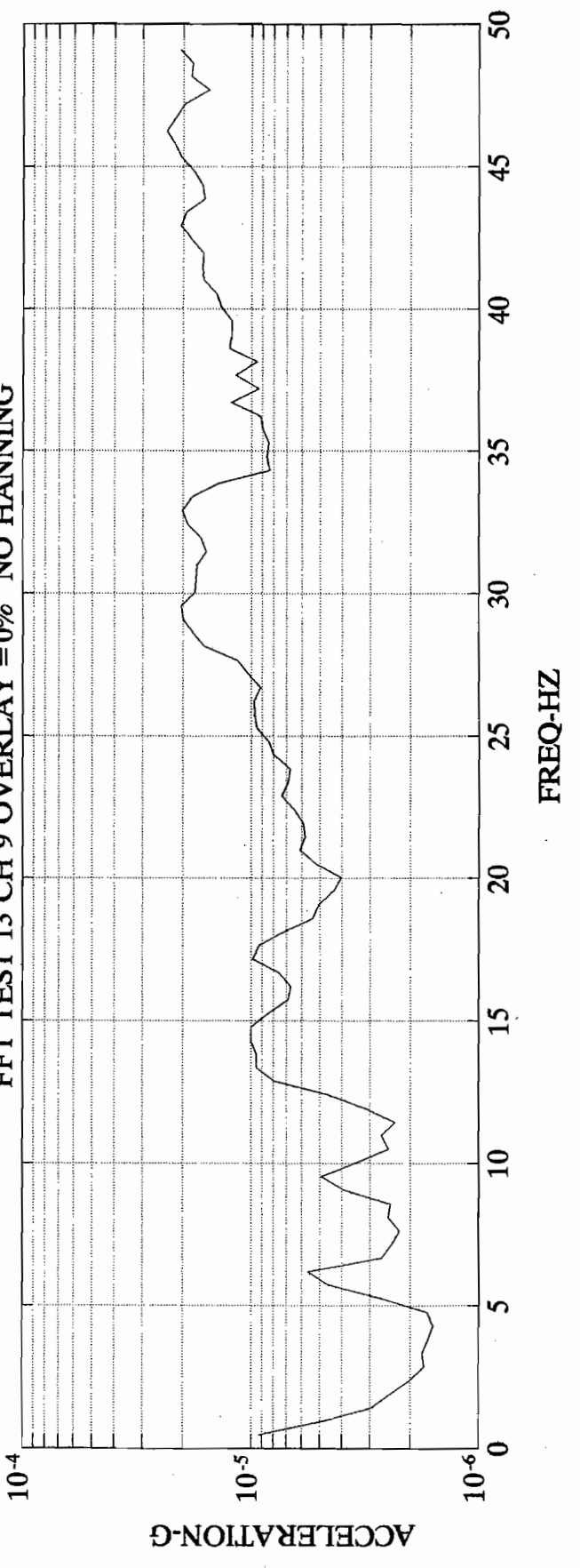
FFT TEST 13 CH 15 OVERLAY =0% NO HANNING

C60

093
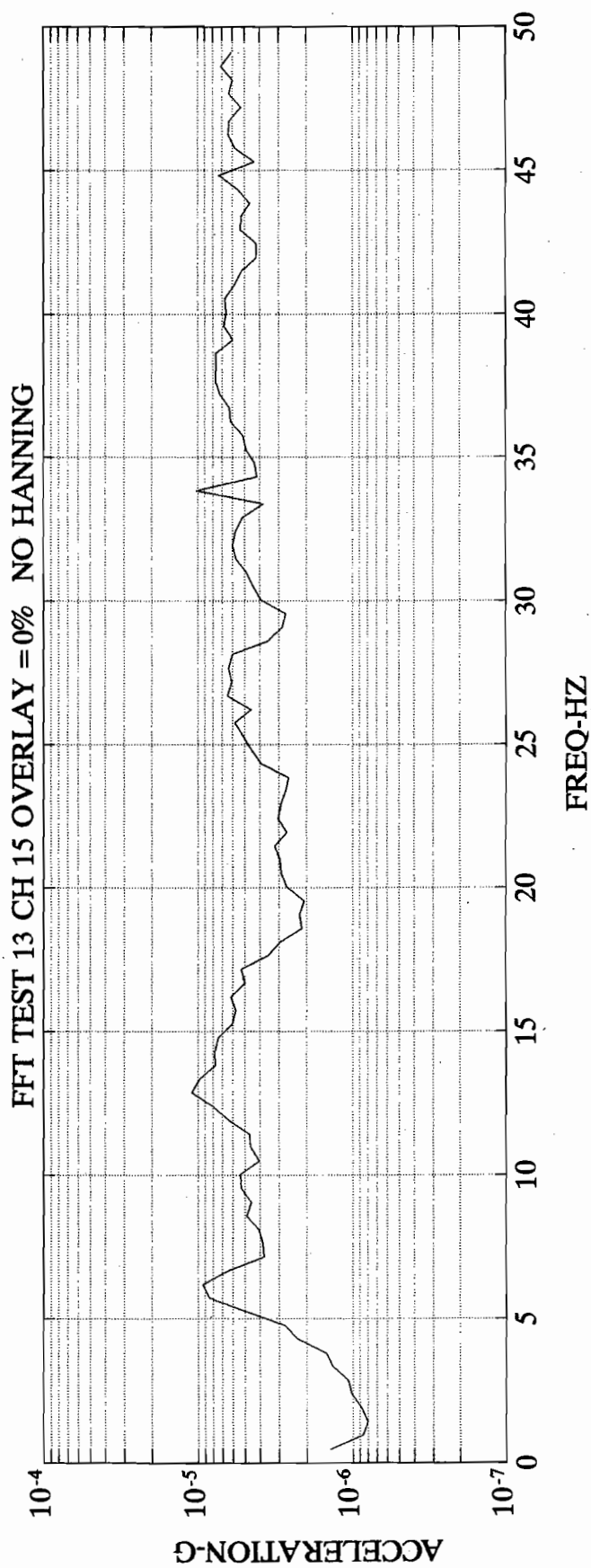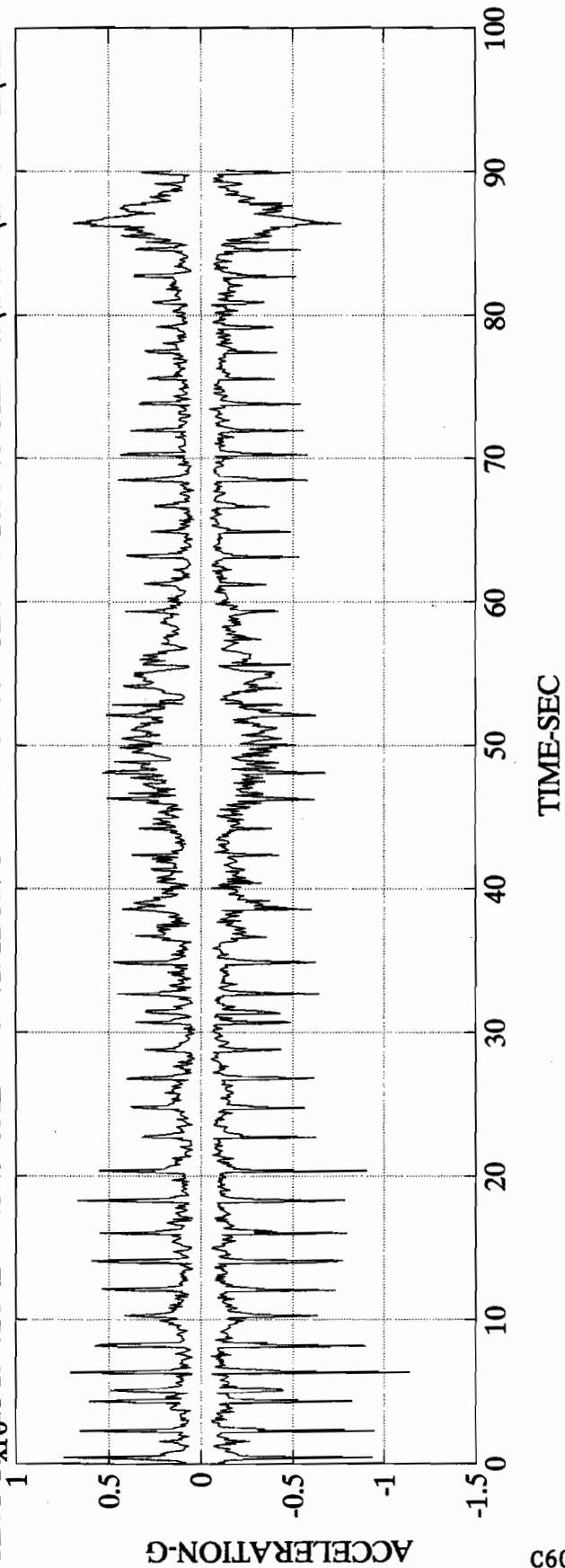
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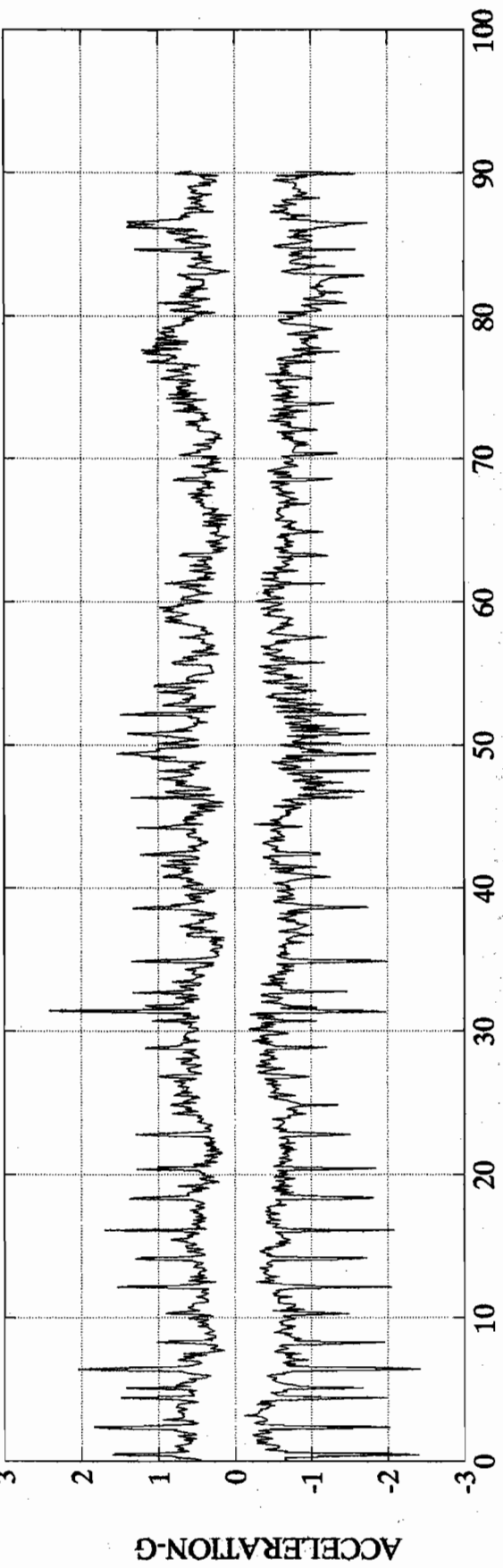
FFT TEST 13 CH 14 OVERLAY =0% NO HANNING

C59
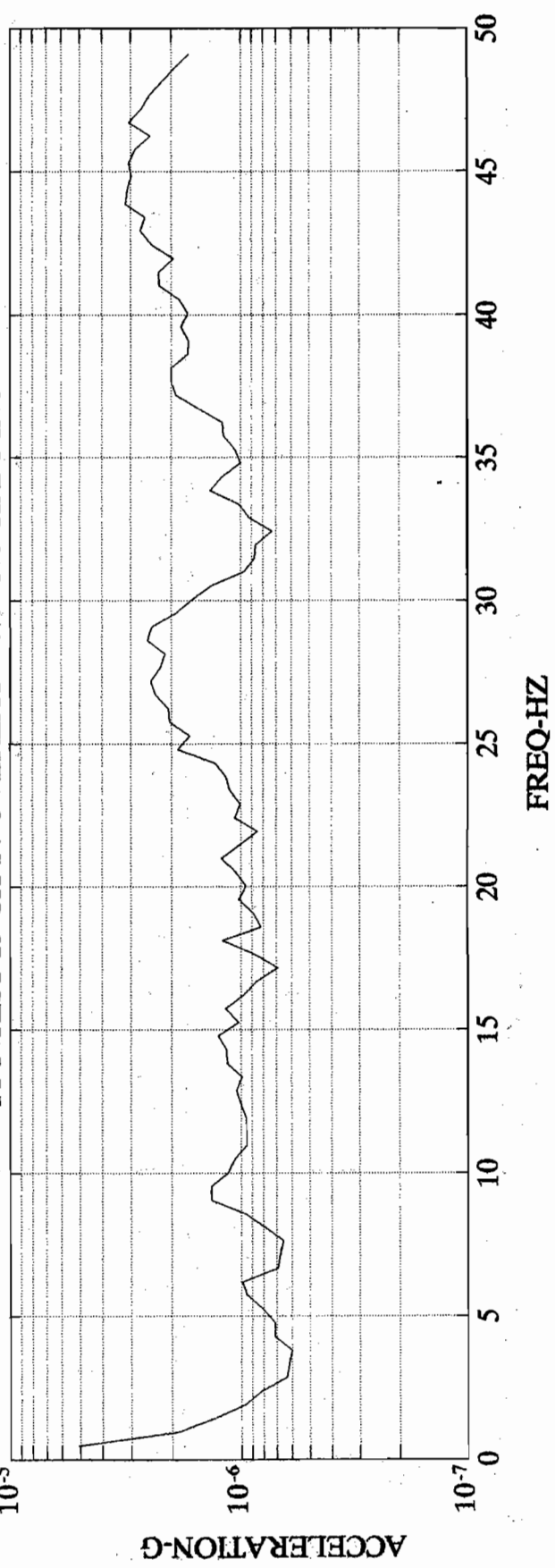
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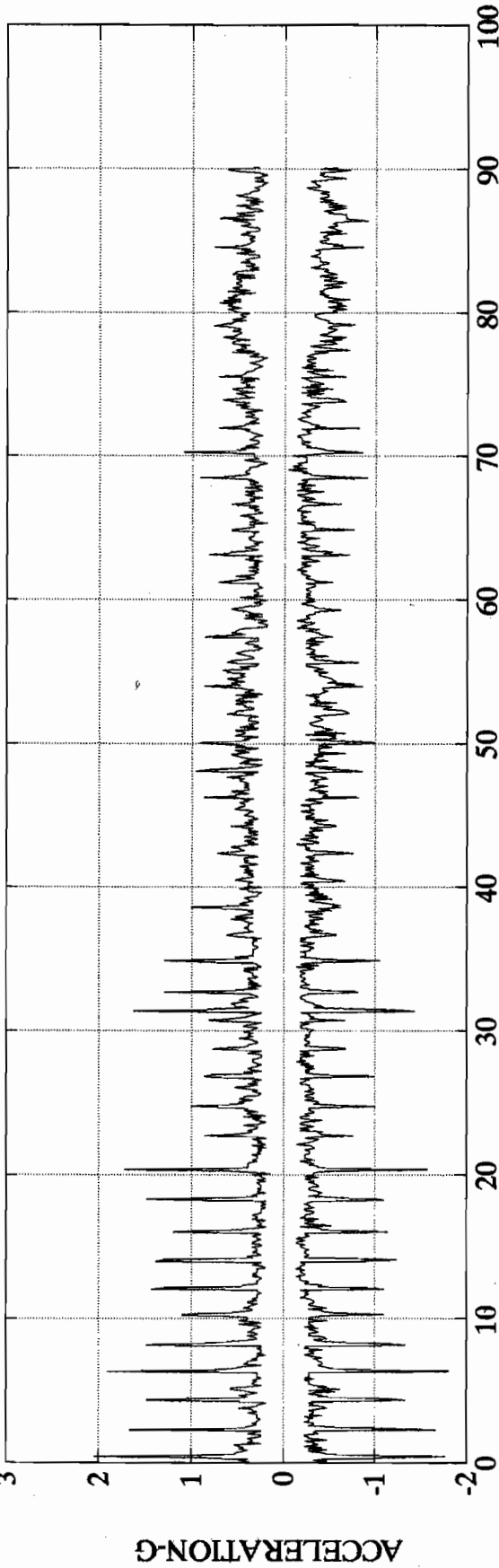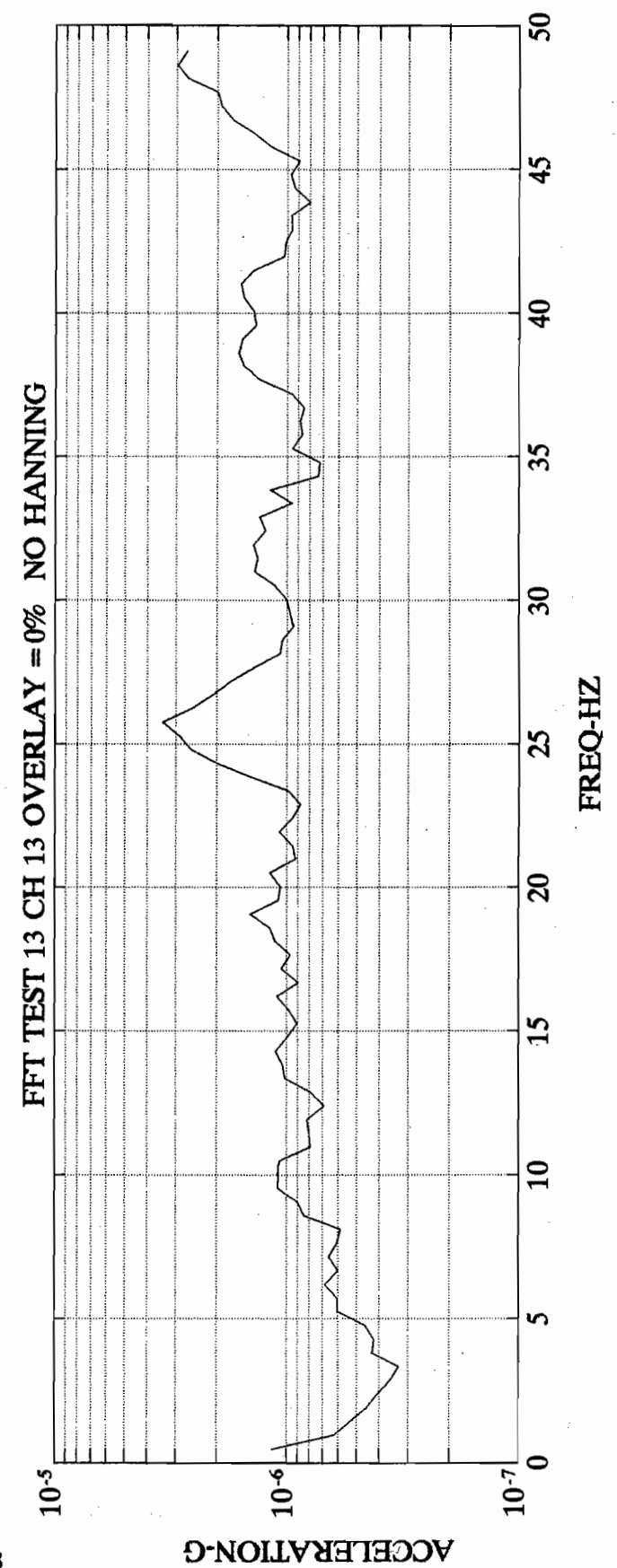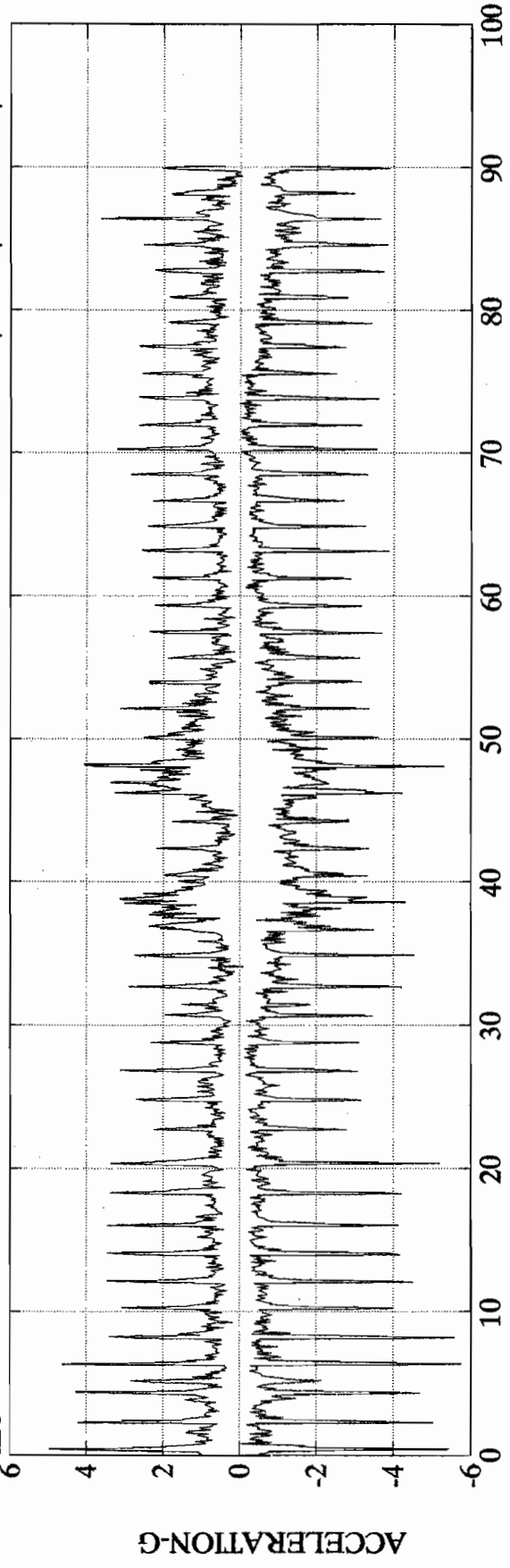

FFT TEST 13 CH 13 OVERLAY =0%   NO HANNING

C58

TEST 13 CH 12 RATE = 488.3 CAL = -0.00189 AVG = 3.567e-06 VAR = 3.235e-09 FILE: c:\ateam\newateam\hm0515(

ACCELERATION-G

TIME-SEC

FFT TEST 13 CH 12 OVERLAY = 0%   NO HANNING

ACCELERATION-G

FREQ-HZ

C57
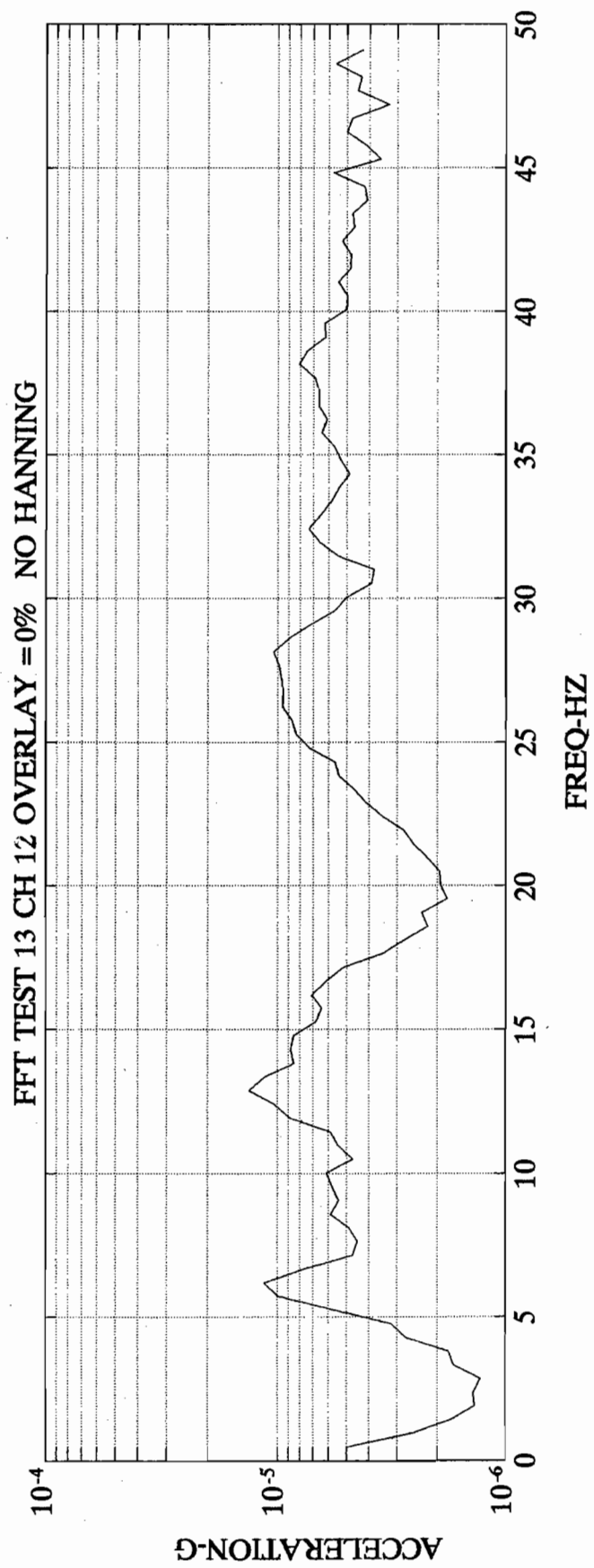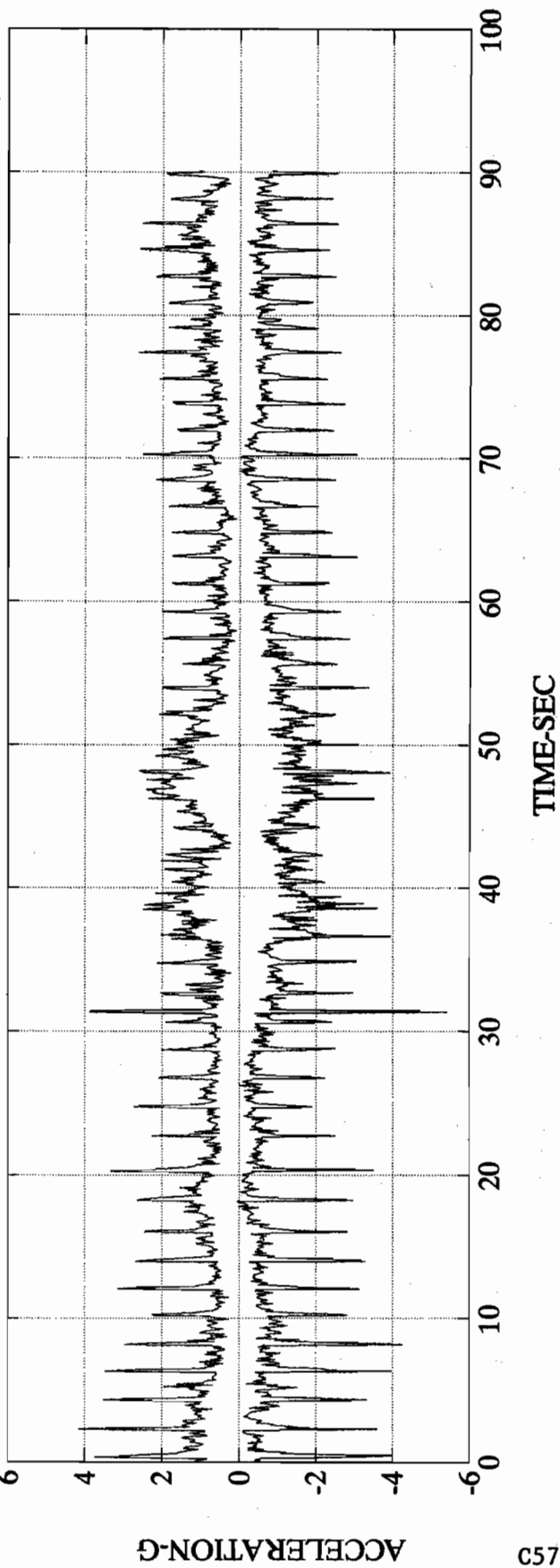
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FFT TEST 13 CH 11 OVERLAY =0% NO HANNING

C56
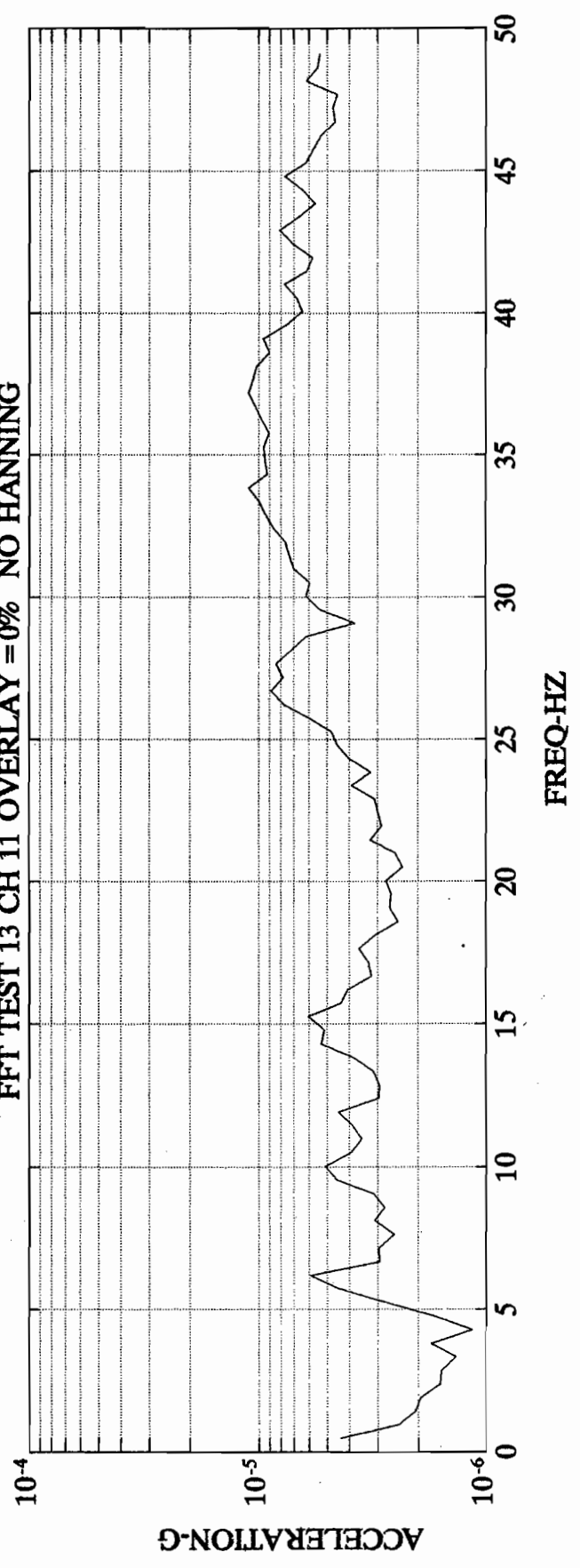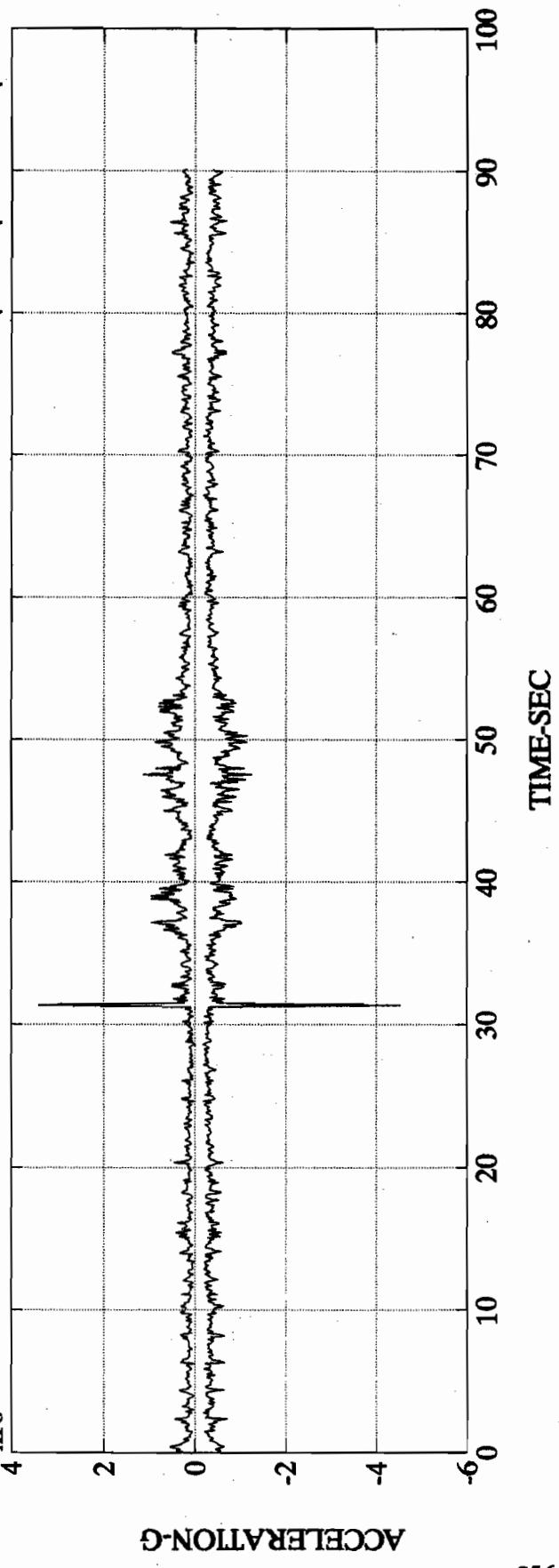
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ACCELERATION-G

TIME-SEC
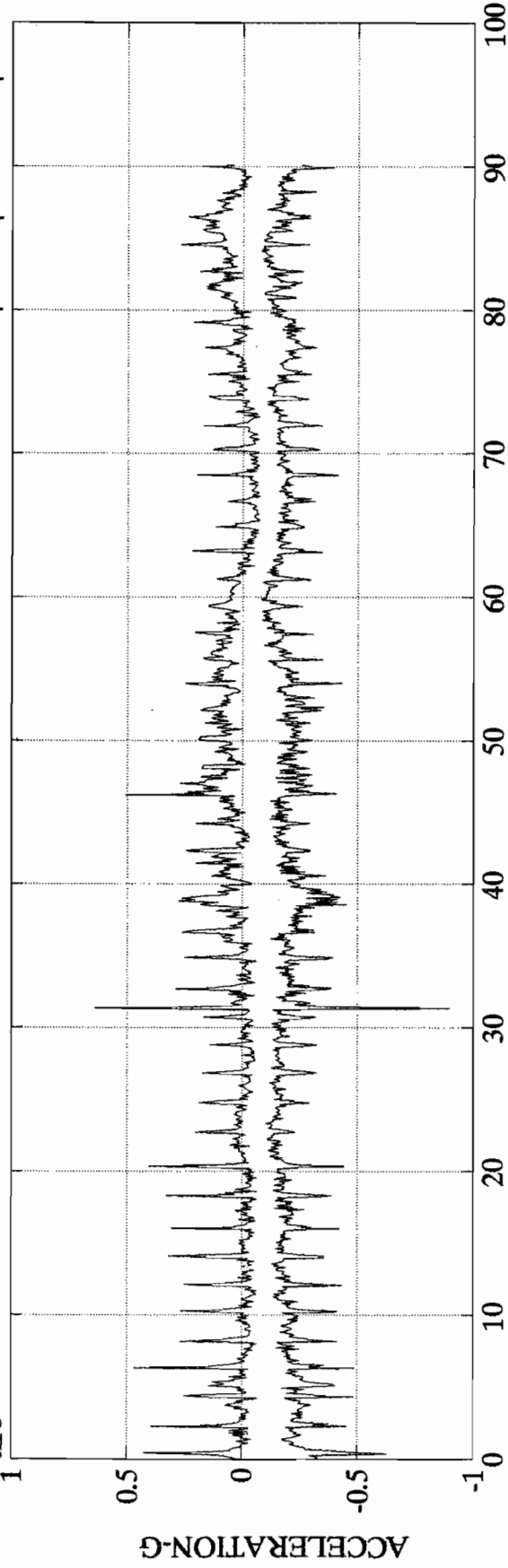
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ACCELERATION-G

FREQ-HZ

C55
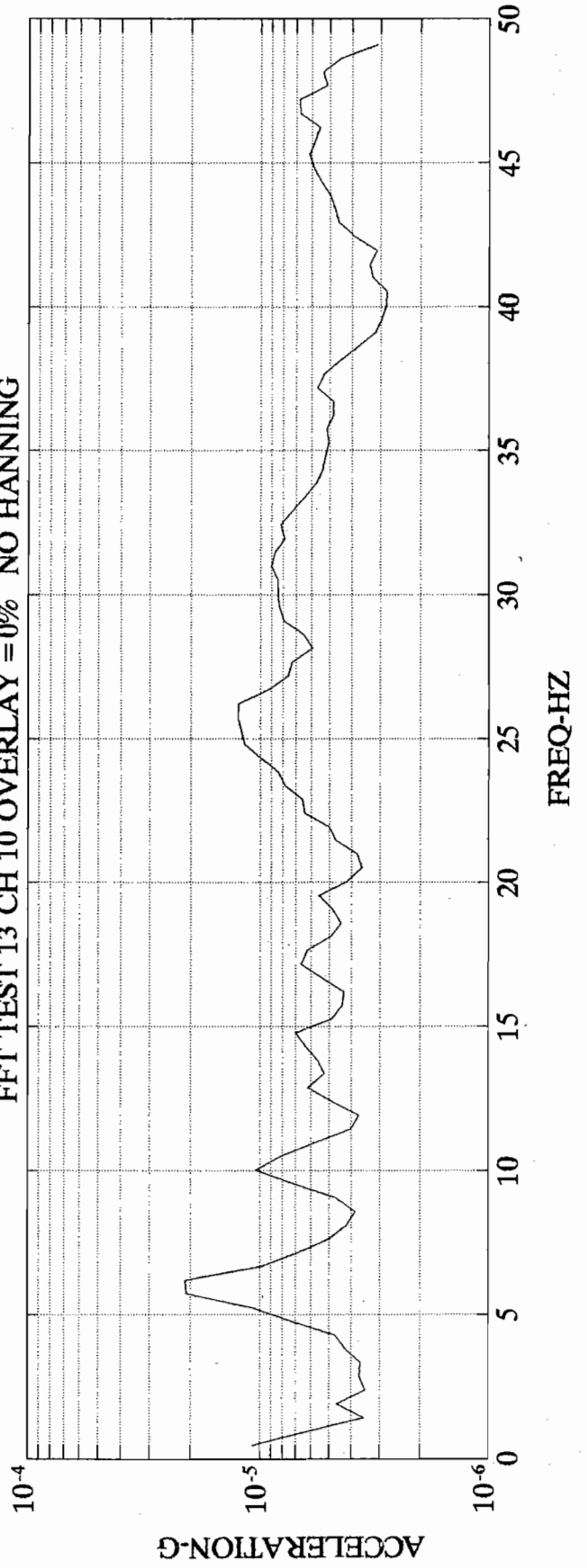
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ACCELERATION-G

TIME-SEC
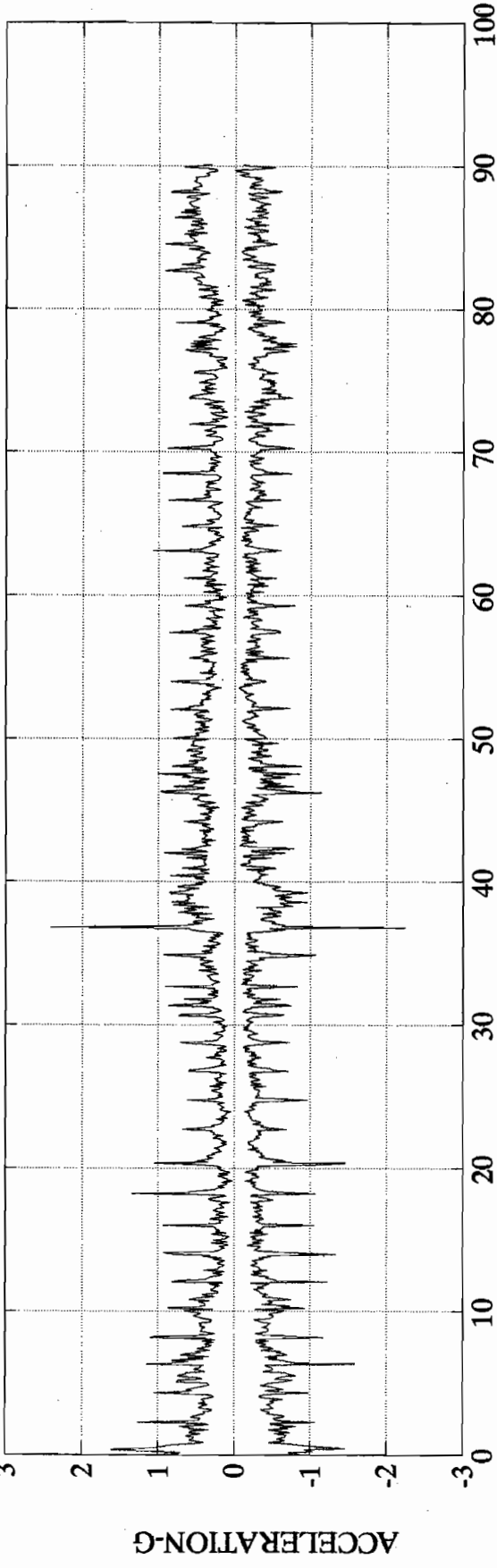
FFT TEST 13 CH 9 OVERLAY =0% NO HANNING

ACCELERATION-G

FREQ-HZ

C54

TEST 120GH 8 RATE = 488.3 CAL = 0.001936 AVG = 5.11e-07 VAR = 4.469e-09 FILE: c:\ateam\newateam\hm051500.

TIME-SEC

ACCELERATION-G

FFT TEST 13 CH 8 OVERLAY = 0%   NO HANNING

FREQ-HZ

ACCELERATION-G

C53
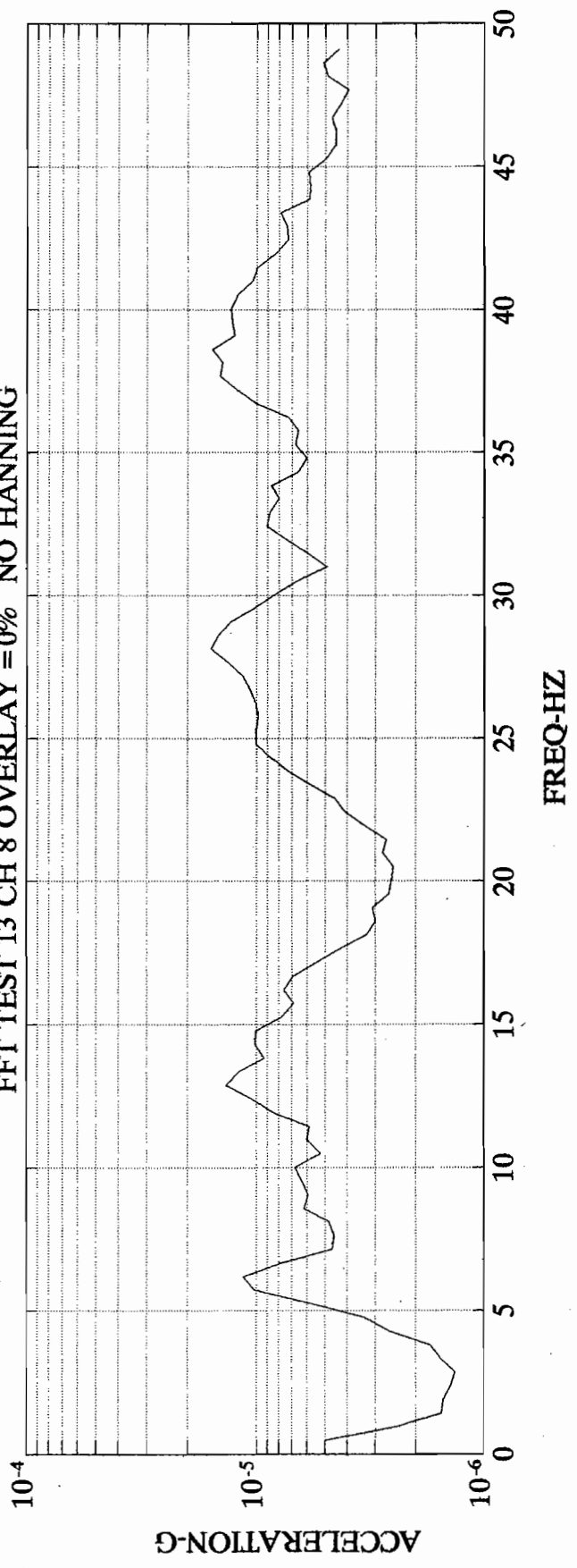
APPENDIX D

2-D Static Analyses of Floor Slab and Basement Wall

Appendix D
2-D Static Analyses of Floor Slab and Basement Wall

Problem: Determine the deflection of one-way slab (2-D slice) of an unreinforced concrete slab at point of incipient cracking.
Formulation:
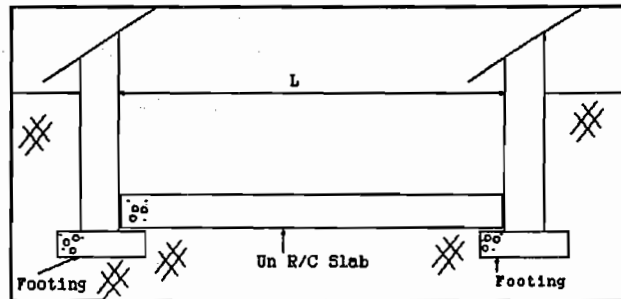
1. Assume linear elastic 2-D response of unreinforced concrete (Un R/C) slab.
2. Thickness, t, is not greater than 1/10 of span length, L.
3. The cracking strength of concrete is given by ACI 318-89 (American Concrete Institute, 1989), f_r, the modules of rupture, (in. psi):

$f_r = 7.5 \ (f_c')^{1/2}$

(equation 9-9 from 9.5.2.3 of ACI 318-89)

where f_c' = specified compressive strength of concrete, psi

4. f_c' = 3,000 psi

$f_r = 7.5(3,000)^{1/2} = 410.8$ psi, say 411 psi (from equation above

f_r = 230-400 psi (from Figure 3.2)

5. Assume as a worst case the slab acts as a simply supported beam such that a center load is acting upward and causes a deflection, δ. Then from Popov (<u>Mechanics of Materials</u> 1976, p 130) the maximum tensile stress is given by:

$$\sigma_{max_t} = \frac{Mc}{I}$$

where M is the maximum moment at L/2 = PL/4, and where P is the applied load. Also, P can be related to an elastic deformation, δ, such that

D2
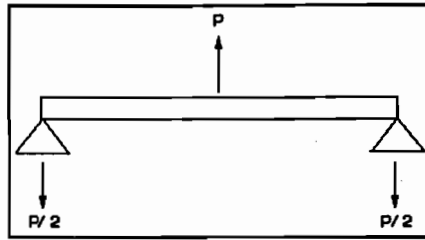
$$\delta = PL^3/48EI$$

where L is the clear span length between end supports, and E is the elastic modulus of concrete (see page 580). E is given by (ACI 318-89, 8.5.1).

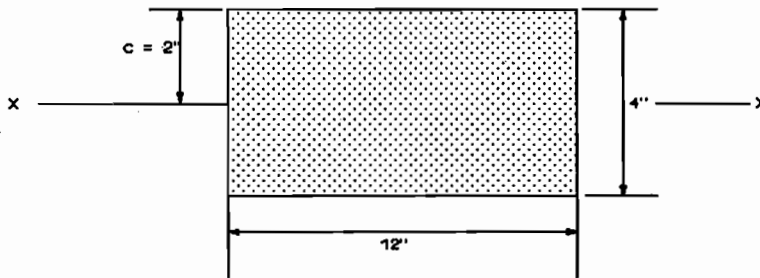$E = 57,000(f_c')^{1/2}$, for normal weight concrete (in. psi)

$E = 57,000(3000)^{1/2}$

$E = 3.122 \times 10^6$ psi

and I = the moment of inertia of the cross section of the 2-D slice of the floor slab. Taking a 12 in. slice gives

$I_{xx} = 4^3 * 12/12.$

$I_{xx} = 64$ in.^{4} (about the x-x axis, the center of the cross section).

Thus, setting $\sigma_{max} = f_r$

gives $f_r = [(PL/4)(c)]/I$

solving for P, gives

$P = [(4)(f_r)(I)]/Lc$, let L=40 ft span (typical short dimension of slabs in the houses visited).

$P = [(4)*(400 \text{ psi})*(64 \text{ in.}^4)]/[(40\text{ft}*12\text{in./ft})*(2\text{in.})]$

$P = 107$ lb ($f_r = 400$)

$P = 61$ lb ($f_r = 230$)

$\delta = [(107 \text{ lb})(40 \text{ ft} * 12 \text{ in./ft})^3]/[(48)(3.122\times10^6 \text{ psi})(64 \text{ in.}^4)]$

$\delta = 1.2$ in. ($f_r = 400$)

$\delta = 1.2 (230/400) = .7$ ($f_r = 230$)

Problem: Determine linear elastic stresses in a 2-D section of an unreinforced masonry block (UMB) wall.

Formulation:
1. Assume the deformations are small relative to the height of the wall (<1.5% max)
2. The mortar and the blocks have essentially the same modulus up to cracking in the mortar.
3. From discussions with Hadala, a possible scenario for wall pressures is a uniform pressure.

   Active pressure = 400 lb-ft/8 ft = 50 psf

   Swelling pressure = 3750 lb-ft/8 ft = 470 psf

4. Assume the wall is 8 ft high and 20 in. wide and the

effective house load on a 1-ft section is 1,000.
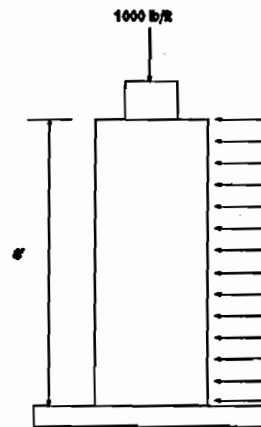
Then the maximum stress

$\sigma_{max} = Mc/I$

   M_{active} = (50 psf)(8 ft)(1 ft)(4 ft) = 1.6 K-FF = 19.2 K-in.

   $M_{swelling}$ = (470 psf)(8 ft)(1 ft)(4 ft) = 15 K-FF = 180 K-in.

   (1 K = 1,000 lb)

   σ_{active} = (19.2 K-in.)(10 in.)/80000 in.^{4} = 24 psi
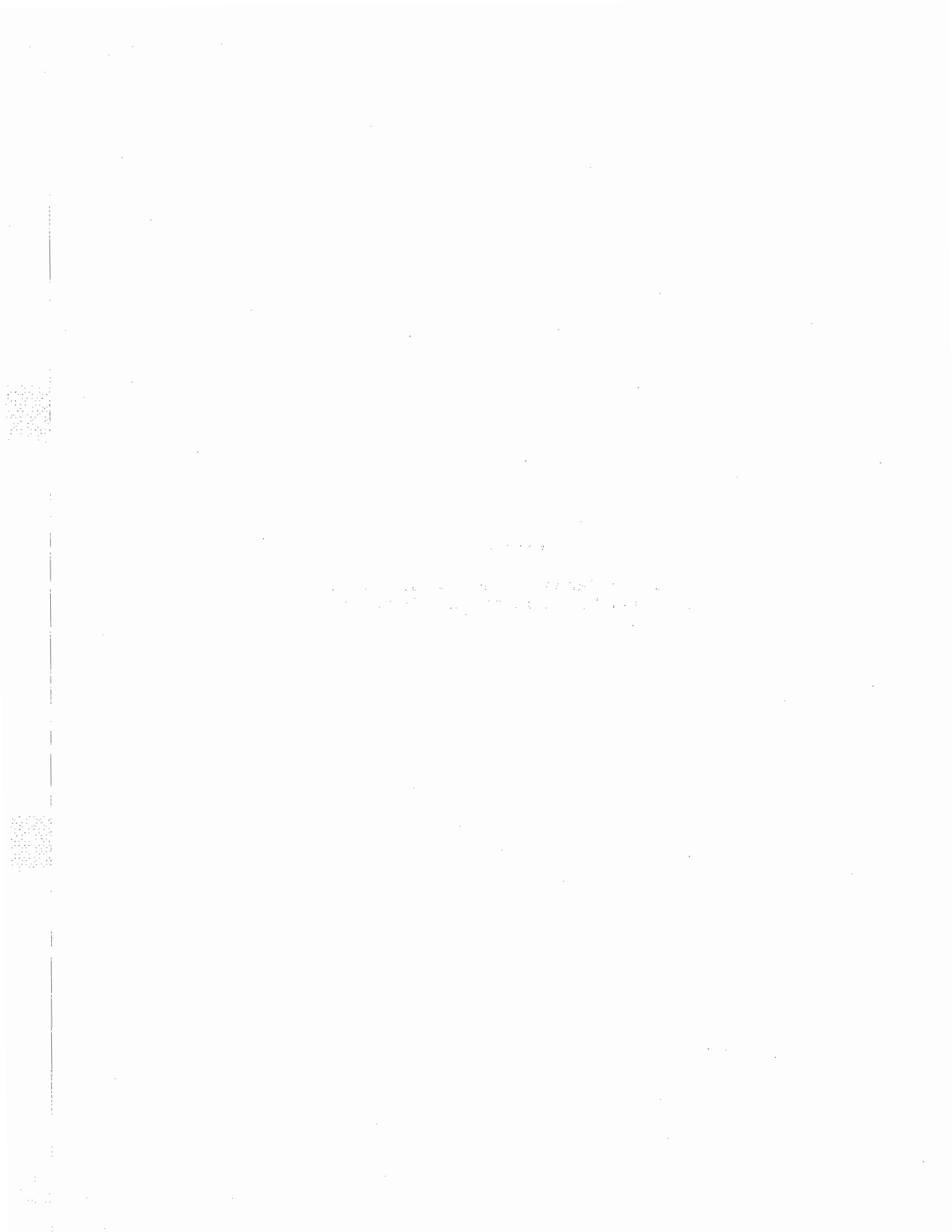

   $\sigma_{swelling}$ = (180 K-in.)(10)/8000 = 225 psi

adding stress due to load of house

D4

$\sigma_H = 1,000/[(20)(12)] = 4.2$ psi

Maximum tension is

$\sigma_{active} = 24-4.2 = 19.8$ psi

$\sigma_{swelling} = 225-4.2 = 220$ psi

Appendix E

Fatigue Damage to Homes in Daylight
and McCutchanville, Indiana

M E M O R A N D U M

TO: Dr. Robert Hall, CEWES-SS-A

FROM: Dr. Sam Kiger

DATE: 10 July 92

SUBJECT: Fatigue Damage to Homes in Daylight and McCutchanville, Indiana Potentially Related to Surface Mine Blasting.

I have reviewed the references listed in enclosure 1 to assess the probability that the subject homes were damaged by blast induced vibrations cumulatively, e.g. by material fatigue (resulting in material failure) from repetitive blast vibration events.

Siskind, et. al. in Reference 11 indicates that 5 to 10 in/sec (ips) peak particle velocity (ppv) blast vibrations is the minimum ppv required to crack concrete walks, driveways and foundations, and to cause major superstructure cracks. The highest ppv recorded in the Reference 11 study is 0.1 ips. However, Hadala indicates in Reference 1 that values of about 5 times this number could have occurred in the vicinity of Daylight and maximum values of 0.2 ips could have occurred near McCutchanville. Thus, the maximum predicted values of ppv are at least an order of magnitude lower than the minimum ppv to cause major damage. At this low level of vibration, I do not believe the major damage observed, e.g. cracking of basement floors and driveways, can be attributed to material fatigue failure. In any case, those structural elements that are loaded in compression, such as basement walls, will not fail in fatigue. The possibility that these small amplitude repeated ground vibrations may result in consolidation of the foundation soil leading to differential settlement and major structural damage is being investigated by Dr. Paul hadala, U.S. Army Waterways Experiment Station.

Threshold damage is defined as superficial hairline cracks in plaster that can be seen. The U.S. Bureau of Mines Report RI 8896 dated 1984 (Reference 5) documents a study of repeated blast vibration on a wood frame home. On page 55 they indicate wallboard joint cracking after 56,000 cycles at a ppv of 0.5 ips. They go on to say that this corresponds to 2 blasts per day at 5 cycles per blast at a ppv of 0.5 ips for 28 years. Note that 0.5 ips was the

maximum predicted ppv by hadala in Reference 1. Ralston, Director of Indiana Department of Natural Resources (INDR), in his memorandum dated May 10, 1991, indicated the ppv recorded in McCutchanville by INDR was about 0.03 ips in 1987 and was still 0.03 ips in 1991. Thus, it seems that the average ppv is at least an order of magnitude less than 0.5 ips. Dowding (Reference 6, Figure 11.6, page 161) indicates that 88 percent of gypsum panels will not crack before 100,000 cycles at 50 percent of their static strength. He goes on to say that similar tests conducted on plasterboard confirmed this trend. Appendix A in Reference 5 list failure strains for gypsum wallboard in bending as varying between about 900 and about 4700 μ in/in. Figure 25 in Reference 5 relates maximum ground vibration to wallboard strain, and indicates the maximum strain at a ppv of 0.5 ips is less than 100 μ in/in. Again, this is at least an order of magnitude less than the level required to cause fatigue damage.

All of the data I reviewed indicates that the maximum ppv recorded at the subject homes was not greater than 0.5 ips, and the average ppv was at least an order of magnitude less. Therefore, it is very unlikely that cosmetic damage was caused by fatigue failure from repeated blast produced ground vibrations. In conclusion, it is my opinion that damage observed in the subject homes is not the result of material fatigue failure from repetitive blast vibration events.

REFERENCES

1. Hadala, P.F. "Inspection of Building Damage Near Daylight and McCutchanville, Indiana, and Examination of Related Documents." Memorandum For Record, CEWES-GV-Z (70-1x), Vicksburg, MS, 11 Mar 91.

2. Chiarito, V.P. "Trip Report: Meeting and Field Study on Residential Structural Damages Potentially Related to Surface Mine Blasting In Vanderburgh, County, 19-21 Feb 1991, Observation and Recommendations." USAEWES-SS-R, Vicksburg, MS, March 1991.

3. Siskind, D.E., Stagg, M.S., Kopp, J.W. and Dowding, C.H. "Structure Response and Damage Produced by Ground Vibration from Surface Mine Blasting." R.I. 8507, U.S. Bureau of Mines, 1980.

4. Siskind, D.E., Stachura, V.J., Stagg, M.S., and Kopp, J.W. "Structure Response and Damage Produced by Airblast From Surface Mining." R.I. 8485, U.s.Bureau of Mines, 1980.

5. Stagg, M.S., Siskind, D.E., Stevens, M.G., and Dowding, C.H. "Effects of Repeated Blasting on a Wood-Frame House." R.I. 8896, U.S. Bureau of Mines, 1984.

6. Dowding, C.H. "Blast Vibration Monitoring and Control." Prentice Hall, Inc., Englewood Cliffs, NJ, 1985.

7. Wiss, J.F. and Nichols, H.R. "A Study of Damage to a Residential Structure From Blast Vibrations." American Society of Civil Engineers, New York, NY, 1974.

8. Bhattacharya, S., Baker, C.H., and Huck, P.J. "Richey Investigation Report." Engineers International, Inc., Westmont, IL, 10 Feb 86.

9. Woods, R.D. "Transmission of Blast Induced Seismic Waves in Vicinity of McCutchanville, Indiana." Undated.

10. Woods, R.D. Letter to David Pope, dated 3 Oct 1990, regarding Amax Coal, McCutchanville, and Daylight, Indiana.

11. Siskind, D.E., Crum, S.V., and Plis, M.N. "Vibration Environment and Damage Characterization for Houses in McCutchanville and Daylight, Indiana," Contract Research Report, Interagency Agreement EC68-IA9-13259, U.S. Department of the Interior, Bureau of Mines, Twin Cities Research Center, 1990.

12. Siskind, D.E., Crum, S.V., Otterness, R.E., and Kopp, J.W. "Comparative Study of Blasting Vibrations from indiana Surface Coal Mines." R.I. 9226, U.S. Bureau of Mines, 1989.

E4

13. Ralston, P.R., Director, Indiana Department of Natural Resources, Letter to Mr. Carl Close, Office of Surface Mining, Pittsburgh, PA, May 10, 1991.

CHAPTER 3

STATIC SETTLEMENT ANALYSES

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
3909 HALLS FERRY ROAD
VICKSBURG, MISSISSIPPI 39180-6199

CEWES-GV-A

30 Dec 92

MEMORANDUM FOR RECORD

SUBJECT: Settlement and Bearing Capacity Calculations Based on Soil
Property Data Collected in the Daylight and McCutchanville Areas

Introduction

1. In reference 2.a., (the interagency agreement between WES and OSM), WES
was to "conduct engineering analysis to (1) estimate vertical wall loadings on
footings, (2) determine probable extent of foundation settlement from esti-
mated static wall loads, and (3) determine differential settlements required
to cause yield line cracking in unreinforced basement floor slabs."

This memo addresses item (2) and summarizes bearing capacity and settlement
calculations performed by the undersigned and attached as encl 1. Items (1)
and (3) are the subject of separate studies by the Structures Laboratory (SL),
WES.

References

2. The following data sources and references were used:

a. "Interagency Agreement between the Office of Surface Mining Reclama-
tion and Enforcement, US Department of The Interior and Corps of Engineers
Waterways Experiment Station, Field and Laboratory Evaluation of Potential
Causative Factors of Structural Damages in Daylight/McCutchanville, IN," 1991.

b. Stephenson R. J., CESAD-EN-FL letter report dated 12 July 1990 to
Mr. Bob Welsh, Office of Surface Mining.

c. Maynard B. R., letter to P. F. Hadala dated 24 July 1992, transmitting
results of Contract No. EF-68-RFP-92-12008, Office of Surface Mining,
Pittsburgh, PA.

d. Memorandum for Record, dated 30 December 92, subject: Laboratory Soil
Testing - Interagency Agreement No. EF68-IA91-13796, "Field and Laboratory
Evaluation of Potential Causative Factors of Structural Damages in Daylight/
McCutchanville, IN."

e. Terzaghi, K. and Peck, R. B. "Soil Mechanics In Engineering Practice,"
2nd Edition, 1967, John Wiley & Sons Inc., New York, NY.

ROUTING:
1. ~~CEWES-GV-Z (Dr. Marcuson)~~
2.
3. CEWES-GV-A (Mrs. Staer - file)

| HYDRAULICS | GEOTECHNICAL | STRUCTURES | ENVIRONMENTAL | COASTAL ENGINEERING | INFORMATION |
| LABORATORY | LABORATORY | LABORATORY | LABORATORY | RESEARCH CENTER | TECHNOLOGY LABORATORY |

-52-

SUBJECT: Settlement and Bearing Capacity Calculations Based on Soil
Property Data Collected in the Daylight and McCutchanville Areas.

    f.   US Army Corps of Engineers, "Engineering and Design, Settlement
Analysis," EM 1110-1-1904, 30 September 1990, Washington, DC.

    g.   Siskind, D. E., Crum, S. V., and Pels, M. N. "Vibration Environment
and Damage Characterization For Houses in McCutchanville at Daylight,
Indiana," Contract Research Report February 1990, Department of the Interior,
Office of Surface Mining.

Base Case

3.   Mr. Vince Chiarito of SL provided the vertical wall loading base case
based on calculated structure dead loads for 1 (and 2) story residential build-
ings.   The base case was given to the undersigned as a 2100 lb/lineal ft load
on a strip footing 20 inches wide, 4 ft below original ground.   The bearing
pressure is 1260 lb/ft^2 which is not considered a high bearing pressure.   I
assumed a square foundation excavation 4 ft below original ground containing a
50 x 50 ft basement whose walls are supported on the strip footing described
above.

Soil Property Data

4.   References b. and c. contain soil index property data from numerous bor-
ings in the Daylight and McCutchanville area.   Reference d. contains shear
wave velocity data from the area and cites additional USBR shear wave velocity
data from the area.   Reference b. contains the results of 14 consolidation
tests on undisturbed sample from 4.0 to 15.0 ft in depth from several borings
in the area.   Eleven of these tests are on non-swelling CL soils and three are
on swelling CH or CL-CH soils.   All the tests indicate that the soils are pre-
consolidated; that is, each has been subjected to a vertical stress for a long
period of time larger than the current overburden stress.   Possible causes of
this are desiccation of clays or erosion of overburden over a long period of
time.   Pre-consolidation pressures do not exceed 4 kips/sq ft.   Existing over-
burden pressures at the consolidation specimen locations are less than or
equal to 1.5 kips/sq ft.   For erosion to be the cause, erosion of 30 ft of
material would be required over geologic time.   Pre-consolidation by glacial
ice is ruled out because of the low pre-consolidation pressure and geologic
literature which indicates this area is outside the limits of continental
glaciation.   Preconsolidation by desiccation during the deposition process or
by interaction with a root structure (as can occur in a fragiapan) are the
most likely causes.   The low initial void ratios noted in reference d., and
which can also be seen in reference b., are consistent with pre-consolidation.
The consolidation test data are summarized on page 2 of the enclosure.

2

SUBJECT: Settlement and Bearing Capacity Calculations Based on Soil
Property Data Collected in the Daylight and McCutchanville Areas.

Because pre-consolidation pressures are substantially more than the bearing
pressure, one should not expect large settlements.

Bearing Capacity

5.   The bearing capacity of a 20 in. wide strip footing was calculated under
the very conservative assumption that it was a surface strip footing (instead
of being 4 ft deep) using equation 33.6 of reference e. (unconsolidated-
undrained (Q) triaxial test and unconfined compression (UC) test shear
strength data from reference b., standard penetration test blow count data
from reference c., and blow count versus strength correlations from page 347
of reference e.   A factor of safety of 3.0 (conventional practice) was chosen.
The allowable bearing capacity exceeds the estimated 2100 lb/lineal foot load-
ing even when using strength values exceeded by 90 percent of the test data.
See page 1 of the enclosure for the details of the analysis.   Bearing capacity
failure is therefore not a reasonable scenario for the footing size and load
in the base case and the soils encountered in the subsurface investigations.

Settlement

6.   There are several parts to the process of estimating settlements.   They
are: (1) analysis of soil data to determine pre-consolidation pressure (P_C),
compression index (C_C) and rebound compression index (C_R) (see page 2 of
enclosure), (2) calculation of vertical stress increments as a function of
depth under the middle of a footing and at a basement corner due to excavation
of the basement and the addition of the wall loading (see pages 3 and 4 of the
enclosure), (3) calculation of immediate settlement under the footing load
(see page 5 of the enclosure) and (4) calculation of long term settlement
under the footing (page 6 of the enclosure).

7.   Soil Data.   The void ratios, pre-consolidated pressures, and compres-
sibility data are consistent with index property, blow count, and strength
data.   The sites where samples were obtained are pre-consolidated at least
down to depths of 15 ft.   As will be seen later, the footings produce a negli-
gible change in stress in the soil below 11 ft.   It is therefore appropriate
to use a rebound index C_R for settlement calculations.   Measured values of C_R
range from 0.01 to 0.03 and a value of 0.02 was chosen for use in the
analysis.

8.   Vertical Stress versus Depth.   The Boussinesq solution, (reference f.,
page C-6) was used to calculate vertical stress increments in a linear elastic
medium under a corner and the mid point of a 2100 lb/ft loaded strip footing
around the perimeter of a 50 ft square excavation 4 ft deep due to the combi-
nation of the excavation (a decrease in stress) and the loaded strip footing

3

SUBJECT: Settlement and Bearing Capacity Calculations Based on Soil
Property Data Collected in the Daylight and McCutchanville Areas.

(an increase in stress). Two plots on page 4 of the enclosure show the
results. As shown in the plots, there is a net increase in stress in the top
7 ft below the footing. This means that only shallow depth soil properties
have any practical influence on settlements. It should be noted that the
maximum stresses in these plots are less than the measured pre-consolidation
pressures.

9. <u>Immediate Settlement</u>. A theoretical solution based on the theory of elas-
ticity for the elastic settlement under a uniform, infinitely long strip load-
ing on a linear elastic half-space given in reference f. was used to calculate
immediate settlement. Young's modulus was calculated by assuming that
Poisson's Ratio (ν) was equal to 0.5 and using shear wave velocity data from
reference d. to calculate shear modulus. The calculated settlement was
0.03 in.

10. <u>Time Dependent Settlement</u>. The time dependent settlement calculations
are given on page 6 of the enclosure. They are based upon the rebound
compression index (because maximum stress increment is less than pre-
consolidation pressure), a void ratio of 0.6, and the stress versus depth
plots on page 4 of the enclosure. Maximum time dependent settlements of 0.18
and 0.11 in. were calculated for the centerline and corner respectively.

11. <u>Total Settlement, Differential Settlement and Field Observations</u>. The
calculated total settlements for the corner was 0.14 in. and for the center-
line was 0.21 in. The differential settlement calculated was 0.07 in. (about
1/16 in.). Differential settlements were reported for seven houses in the
study area in reference g. and are substantially larger (greater than 1-in.)
than calculated here and indicated that the downhill side of the buildings had
settled relative to the uphill side. Differential settlements larger than
those calculated could be explained by elastic and consolidation settlement
processes only if (1) the soil profile was nonuniform (one side of the build-
ing was founded on or close to rock and the other over at least 20 ft of soil)
and C_R was substantially larger than measured or (2) the pre-consolidation
pressures reported are wrong. There is no evidence to suggest either of these
possibilities and there is a strong internal consistency among the soils data
which contradicts both possibilities.

12. <u>Summary</u>. Differential settlements of 1.1 to 4.9 in. were measured at
seven houses in the subject area. These settlements far exceed that calcu-
lated based on the soil property data, theory of elasticity and theory of
consolidation for a typical foundation geometry and loading (ie. 0.07 in.).

<div align="center">4</div>

SUBJECT: Settlement and Bearing Capacity Calculations Based on Soil
Property Data Collected in the Daylight and McCutchanville Areas.

Reasonable excursions from the typical case could not account for the differ-
ence. Most of the differential settlement observed at these buildings is due
to some other cause.

PAUL F. HADALA
Assistant Director
Geotechnical Laboratory

Encl

5

1. ALLOWABLE BEARING CAPACITY OF STRIP FOOTING

$$P_a = q_a \cdot B = \frac{q_u}{3} B = \frac{5.2 c B}{3} = 1.7 c B \cdots$$

P_a = allowable load/unit length
q_a = allowable bearing capacity/unit length = $q_u \div 3$
B = footing width
q_u = ultimate bearing capacity
c = apparent cohesion from Q or UC test
γ = moist unit weight of soil = 100 lbs/ft³

Assumptions
   Terzaghi B.C. Theory for Strip footings
   Factor of Safety = F.S. = 3.0
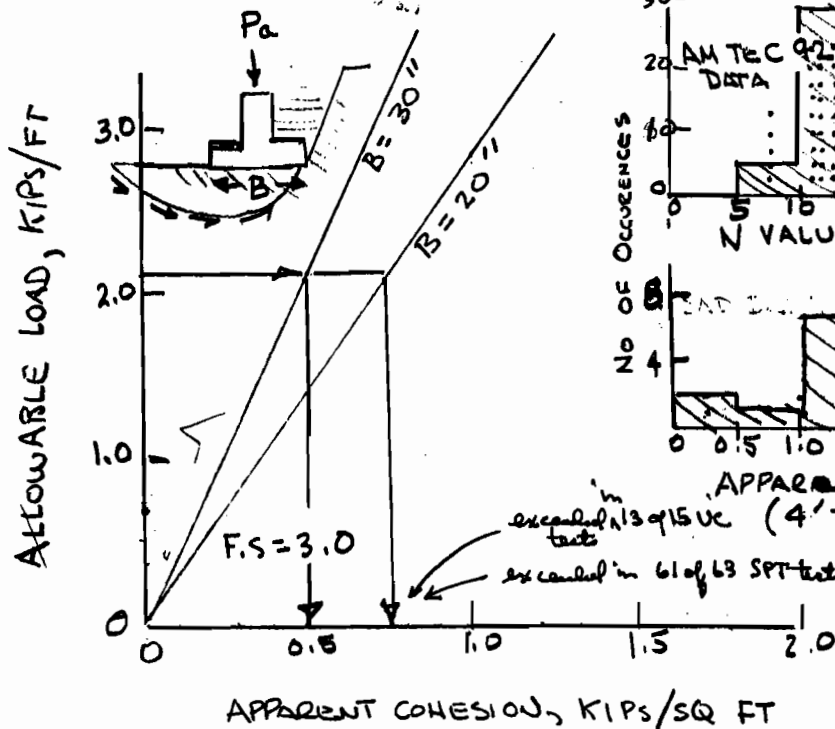   Undrained loading, immed. post construction
   $\phi = 0$ = apparent friction angle (this is
        appropriate for the undrained loading case)
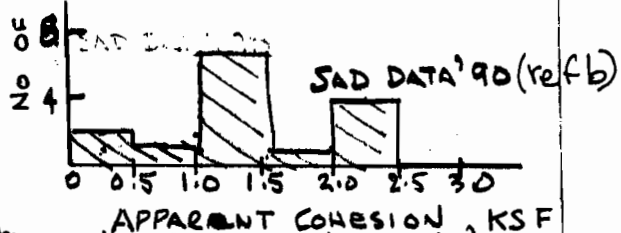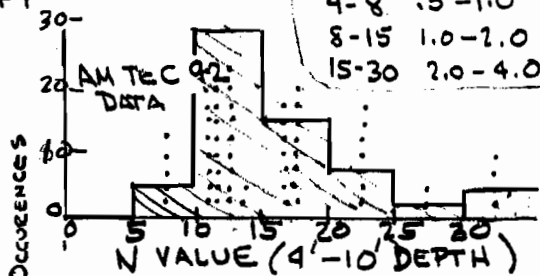
pg 347 ref e

N	C (KSF)
4-8	.5 - 1.0
8-15	1.0 - 2.0
15-30	2.0 - 4.0

$B = 20'' = 1.67$ ft $B = 30'' = 2.5$ ft

2100 LB/LF IS CLEARLY AN ALLOWABLE BEARING
LOAD ON A 20-IN STRIP FOOTING

2. SETTLEMENT COMPUTATION FOR STRIP FOOTING (T&P 68-73)
(ASSUME CLAYS ARE NORMALLY CONSOLIDATED)

This is where $P_0 + \Delta P$ will occur in these mat'ls for residential footing loads

lab meas consol. curve

$$e = e_0 - C_c \log_{10}\left(\frac{P_0 + \Delta P}{P_0}\right)$$

$$\Delta S = \text{Settlement} = \Delta H \frac{C_c}{1 + e_0} \log_{10}\left(\frac{P_0 + \Delta P}{P_0}\right)$$
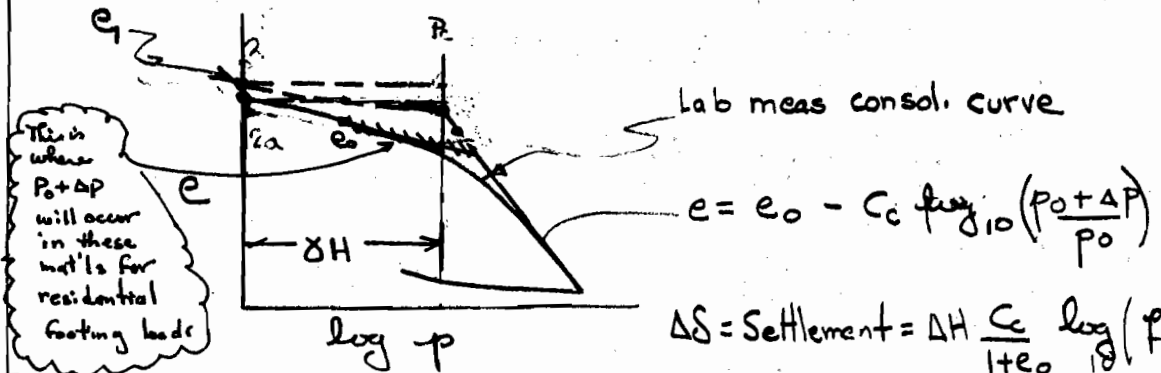
added 30 Dec 92 { e = void ratio, e_0 = in-situ void ratio, P_0 = in-situ stress, Δp = increase in stress due to footing
C_c = compression index, C_R = recompression index, P_c = preconsolidation pressure.

(a) CONSOL TESTS AT DEPTHS OF 4'-20' (SAD 1990 data, ref b)

| Boring	Depth FT	e_1	δH KSF	e_0	e_P	C_c	$\left(\frac{\Delta e}{\log \Delta p}\right) = C_R$			
OSM-1	3.4	0.718	0.3	0.710	0.701	.12	0.03	Non-swelling,	CL	$p_c = 3.2$ KSF
OSM-5	4.0	0.479	0.4	0.470	0.462	.08	0.01	Non-swelling,	CL	$p_c = 3.0$ KSF
OSM-10	5.0	0.030						Swelling	CH	
OSM-12	10.0	0.676	1.0	0.650	.638	0.20	0.02	Non-swelling	CL	$p_c = 3.8$ KSF
OSM-P-17	4.0	0.667	0.4	0.654	.643	.20	0.02	Non-swelling	CL	$p_c = 3.2$ KSF
OSM-P-17	10.0	0.70						Swelling	CL-CH	
OSM-P-17	15.0	0.650	1.5	0.630	.626	.14	0.01	Non-swelling	CL	$p_c = 3.4$ KSF
OSM-29	4.0	0.631	0.4	0.628	.610	.15	0.01	Non-swelling	CL	$p_c = 3.8$ KSF
OSM-29	10.0	0.677						Swelling	CH	
OSM-30	4.0	0.599	0.4	0.595	.586	.09	0.01	Non-swelling	CL	$p_c = 3.6$ KSF
OSM-32	4.0	0.582	0.4	0.578	.570	.09	0.02	Non-swelling	CL	$p_c = 3.4$ KSF
OSM-32	10.0	0.445						Swelling	CL-CH	
OSM-33	4.0	0.415	0.4	0.412	.407	.07	0.01	Non-swelling	CL	$p_c = 1.6$ KSF
OSM-35	4.0	0.562	0.4	0.555		.10	0.01	Non-swelling	CL	$p_c = 3.2$ KSF

These specimens are all overconsolidated to at least twice overburden stress

$e_0 = 0.62$ $e_P = 0.61$ $p_c = 3.$ KSF $C_{c\text{ effective}} \le .03$, best value = 0.02
$\frac{\Delta e}{\log \Delta p} = 0.02$ $C_c \doteq 0.20$

(b) VERTICAL STRESS VS DEPTH (BOUSINESQ)
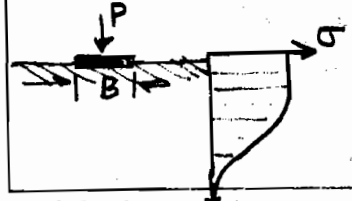
EM 1110-1-1904, Fig C-2, Pg C-6

Stress under center of a 20" wide × 50ft long wall footing loaded to 2100 lb/ft.

Depth z ft	m (B/z)	m (L/z)	I	$\Delta\sigma = 4qI$ KSF	Superposition
0	—	—	0.250	1.30	$q = \frac{2.1}{1.67} = 1.3$ Kips/sq ft
1	0.83	25	.188	0.98	B = ½ width = 0.83 ft
2	0.41	12.5	.117	0.61	L = ½ length = 25 ft
4	0.21	6.3	.063	0.33	
8	0.10	3.1	.031	0.16	
16	0.05	1.6	.016	0.08	
32	0.03	0.8	.006	0.03	
64	0.01	0.4	.002	0.01	

Stress under end of 20" wide × 50ft long wall footing

Depth z ft	m (B/z)	m L/z	I	$\Delta\sigma = 2qI$ KSF	
0	—	—	.50	1.30	q = 1.3 KSF
1	0.83	50	.188	0.49	B = ½ width = 0.83 ft
2	0.41	25	.117	0.30	L = length = 50 ft
4	0.21	12.5	.063	0.16 15	
8	0.10	6.3	.031	0.08	Superposition
16	0.05	3.1	.017	0.04	
32	0.03	1.6	.010	0.03	
64	0.01	0.8	.002	0.01	

Stress relief due to excavation under center of footing

Depth z ft	m (B/z)	m (L/z)	I	$\Delta\sigma = 2qI$ KSF	
0				-.40	$q = 4ft \times \frac{100 lb}{ft^3} = 400$ PSF
1	25	50	0.25	-.20	$= -0.4$ KSF
2	12.5	25	.25	-.20	B = 25 ft
4	6.3	12.5	.25	-.20	L = 50 ft
8	3.1	6.3	.25	-.20	
16	1.6	3.1	.23	-.18	
32	0.8	1.6	.18	-.14	
64	0.4	0.8	.09	-.07	

q = bearing pressure
B = width of loaded area
L = length of loaded area
I = influence factor = $f(m, n)$
z = depth
m = B ÷ z , m = L ÷ z
$\Delta\sigma$ = stress increment

added 30 Dec 92 →

SUBJECT: Settlement Computation for Strip footing and Bearing Capacity COMPUTED BY: Hadala DATE: 2 Sep 92 FILE NO.

CHECKED BY: PFH DATE: 4 Sep 92 SHEET NO. 4

Stress Relief due to excavation under edge of footing

Depth z FT	m (B/z)	m (L/z)	I	$\Delta \sigma = qI$ KSF
0				
1	50	50	.25	−0.10
2	25	25	.25	−0.10
4	12.5	12.5	.25	−0.10
8	6.3	6.3	.25	−0.10
16	3.1	3.1	.25	−0.10
32	1.6	1.6	.23	−0.10
64	0.8	0.8	.15	−0.006

q = −0.4 KSF
B = 50 ft
L = 50 ft

See previous page for symbol definitions

modified 30 Dec 92

$\Delta \sigma_{\mathcal{C}} = -$ STRESS RELIEF + CENTERLINE STRESS

$\Delta \sigma_{COR} = -$ STRESS RELIEF + END STRESS, FOOTING 1 + END STRESS, FOOTING 2

CENTERLINE STRESS, KSF

CORNER STRESS, KSF

net change in stress

≈ 7 FT

(+)

(−)

net decrease in stress is the result of excavation (added 30 Dec 92)

Depth below footing, Ft
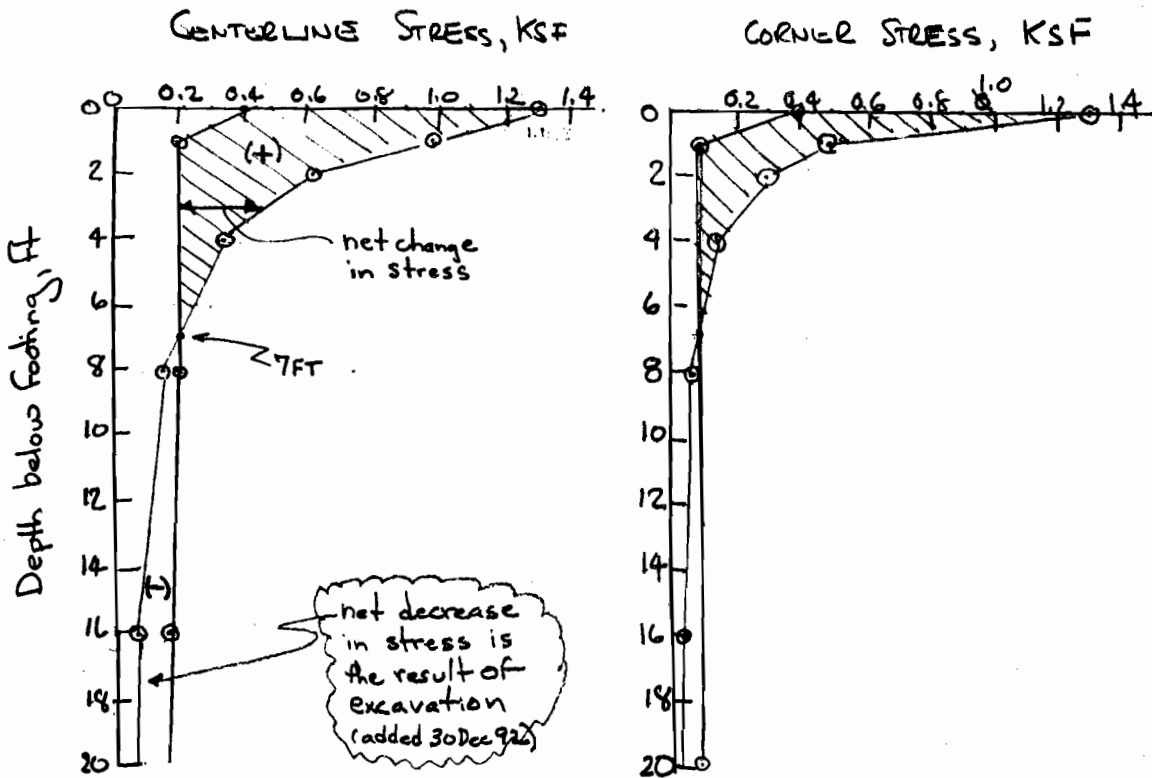
ELASTIC SETTLEMENT CALCULATIONS

EM-1110-1-1904 30SEP90 Pg 3-21 FIG C-6, ref f.

$L = 50'$ $D = 4'$ $H = \infty$ $B = 1.67$
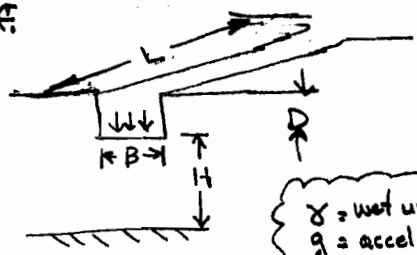
L = length of loaded area
B = width of loaded area
D = depth to loaded area
H = depth to rock

$\dfrac{H}{B} > 10^3$

$D/B = \dfrac{4}{1.67} = 2.4 \Rightarrow \mu_0 = 0.9$

$L/B = \dfrac{50}{1.67} = 30 \Rightarrow \mu_1 = 2.0$

γ = wet unit wt.
g = accel. of gravity
C_s = shear wave velocity

$\rho = \mu_0 \cdot \mu_1 \dfrac{qB}{E_s}$

ρ = elastic settlement
μ_0, μ_1 = influence factors
E_s = Young's Modulus
G = Shear modulus
ν = Poisson's ratio

$E_s = 2G(1+\nu)$, $\nu = 0.5$ $E_s = 3G$

$G = \dfrac{\gamma}{g} C_s^2 = \dfrac{100}{32.2} \dfrac{LB \cdot SEC^2}{FT^2 \cdot FT} (400)^2 \dfrac{FT^2}{SEC^2} = 497,000 \approx 500,000 \text{ PSF}$

$E_s = 3 \times 5 \times 10^5 = 1.5 \times 10^6 \text{ PSF}$

$\rho = 0.9 \times 2.0 \times \dfrac{1300 LB}{FT^2} \cdot \dfrac{1.67 FT}{1.5 \times 10^6} \dfrac{FT^2}{LB} = .0026 FT = 0.03 IN$ ✓

(added 30 Dec 92)

DIFFERENTIAL LEVELING DATA BY USBM

Siskind et al Feb 90 "Vibration Environment and Damage Characterization for Houses in McCutchanville and Daylight, Indiana" — Pgs 63-68, ref g

HOUSE	MAX DIFF. ELEV FT	REMARKS (Survey Acc = 0.01 FT)
105	0.09	
107	0.41	Roof line survey, down slope end is low.
108	0.16	Down slope end is low
209	0.18	" " " " "
215	0.10?	Authors believe this # in error
303	0.27	Down slope end is low
334	0.17	

END VIEW

Consolidation Settlement Calculations

					CENTER		CORNER	
Depth FT	$\Delta\sigma_{\mathcal{L}}$ KSF	$\Delta\sigma_{corner}$ KSF	$\dfrac{C_e \Delta H}{1+e_0}$	p_0 KSF	$\log\left(\dfrac{p_0+\Delta p}{p_0}\right)$	$\Delta\delta$ FT	$\log\left(\dfrac{p_0+\Delta p}{p_0}\right)$	$\Delta\delta$ FT
0	0.90	0.90	0.0125	0.45	0.457	.006	0.388	0.005
	>.84	>0.65			0.320	.004		
1	0.78	0.39	0.0125	0.55	0.189	.002	0.189	0.002
	>.60	>0.30						
2	0.41	0.20	0.0250	0.70	0.142	.002	0.074	0.002
	>.27	>0.13						
4	0.13	0.05	0.0250	0.90	0.046	.001	0.014	0.0004
	>.10	>0.03			0.014	.0004		
6	0.07	0.01	0.0250	1.10	0	0		
	>0.04	>0.005						
8	0	0			$\Sigma = .0013$ ft		$\Sigma = 0.009$	
					$\downarrow 0.18$ in		$\downarrow 0.11$ in	

from pg 4

$e_0 = 0.6 \quad C_{\ell} = 0.02 \quad \Delta H = 1.0 \text{ or } 2.0 \quad \dfrac{0.02 \times 2}{1.6} = 0.025$

$p_0 = 0.1(4+z)$ KSF

(See page 2 for definition of terms)

WHAT VALUE OF $C_{\ell *}$ WOULD YIELD 0.5 IN SETTLEMENT?

$$0.5 = \dfrac{\Sigma \Delta\delta \, C_{\ell *}}{C_R} = \dfrac{\Sigma \Delta\delta \, C_{\ell *}}{0.02} = \dfrac{0.18 C_\ell}{0.02} \quad C_{\ell *} = 0.056$$

THIS VALUE IS BELOW THE OF RANGE OF MEASURED C_c AND IS ... ABOVE THE RANGE OF MEASURED C_R

SUMMARY OF CALC. SETTLEMENT

	$\mathcal{C}$	CORNER
CONSOLIDATION	0.18	0.11
ELASTIC	0.03	0.03
TOTAL	0.21	0.14

FOR A STRIP FOOTING 20 IN WIDE, LOADED TO 2100 LB/LF IN A 4 FT DEEP 50 X 50 FT EXCAVATION, THE ABS MAX SETTLEMENT OF A UNIFORM FOUNDATION MATERIAL WITH PROPERTIES LIKE THOSE ON PG 3 WOULD BE ~ 1/4 IN OR LESS.

DIFFERENTIAL SETTLEMENT WOULD BE ~ 1/16 IN FOR A HOMOGENIOUS FDN AND 1/4 IN OR LESS IF ONE SIDE OF THE FDN. WAS SUPPORTED ON ROCK AND THE OTHER ON AT LEAST 8 FT OF SOIL.

(SEVEN)
NONE OF THE HOUSES SURVEYED HAD DIFFERENTIAL SETTLEMENTS AS SMALL AS THOSE CALCULATED.

WES FORM NO. 1253
REV OCT 1968

CHAPTER 4

EARTH PRESSURE ANALYSES AND MISCELLANEOUS OBSERVATIONS

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
3909 HALLS FERRY ROAD
VICKSBURG, MISSISSIPPI 39180-6199

CEWES-GV-A   9 Jan 93

MEMORANDUM FOR RECORD

SUBJECT: Addendum to 11 Mar 91 Memorandum for Record and Lateral Earth Pressure Analyses for OSM Study

1. Reference is made to:

    a. Memorandum for Record from CEWES-GV-Z (P. F. Hadala) dated 11 Mar 91, subject: "Inspection of Building Damage Near Daylight and McCutchanville, Indiana, and Examination of Related Documents."

    b. Memorandum for Record from CEWES-GV-A (P. F. Hadala and R. W. Peterson) dated 30 Dec 92, subject: "Laboratory Soil Testing - Interagency Agreement No. ET68-IA91-13796, Field and Laboratory Evaluation of Potential Causative Factors of Structural Damages in Daylight/McCutchanville, Indiana."

    c. Memorandum for Record from CEWES-GV-A (P. F. Hadala) dated 30 Dec 92, subject: "Settlement and Bearing Capacity Calculations Based on Soil Property."

    d. Seismological Research Letters, Vol. 59, No. 3, dated Jul-Sep 88, pp. 91-97, article entitled: "A Preliminary Report on the Variability In Particle Velocity Recordings of the June 10, 1987 Southeastern Illinois Earthquake," by R. Street, A. Zekulin, D. Jones, and G. Min.

    e. Memorandum for Record from CEWES-GV-A (P. F. Hadala) dated 25 Nov 91, subject: "Visit to Evansville, Indiana Area."

    f. Trip report by V. Chiarito, subject: "Meeting and Field Study on Residential Structural Damages Potentially Related to Surface Mine Blasting in Vanderburgh County, 19-21 Feb 92, Observations & Recommendations." U.S. Army Engineer Waterways Experiment Station (WES), unpublished report, Mar 91.

    g. Letter (and encls) from B. R. Maynard, Office of Surface Mining, to P. F. Hadala, WES, dated 24 Jul 92.

ROUTING:
1. CEWES-GV-Z (Dr. Marcuson)
2.
3. CEWES-GV-A (Mrs. Staer - file)

| HYDRAULICS LABORATORY | GEOTECHNICAL LABORATORY | STRUCTURES LABORATORY | ENVIRONMENTAL LABORATORY | COASTAL ENGINEERING RESEARCH CENTER | INFORMATION TECHNOLOGY LABORATORY |

CEWES-GV-A   9 Jan 93
SUBJECT: Addendum to 11 Mar 91 Memorandum for Record and Lateral Earth Pres-
sure Analyses for OSM Study

    h.   Book entitled, "Soil Mechanics In Engineering Practice," by
K. Terzaghi and R. B. Peck, second edition, 1967, John Wiley Publishers, New
York.

    i.   Letter (and encls) from C. H. Dowding, Northwestern University to
P. F. Hadala WES dated 30 Mar 1992.

Addendum

2.   Since reference 1.a. was written, I have obtained additional information
that bears on judgments and opinions given therein.

    a.   A WES conducted laboratory study sponsored by OSM and reported in
reference 1.b. eliminated the possibility of blasting vibration induced col-
lapse or pore pressure mobilization as a cause of building settlement of
instability in the Daylight and McCutchanville areas.

    b.   A WES conducted analytical study using existing laboratory consolida-
tion test data for the Daylight and McCutchanville areas reported in refer-
ence 1.c. proved that elastic and consolidation foundation settlements under
typical residential building loads were not sufficient to damage the buildings
and were much smaller than settlements measured at six area buildings by the
USBM.

    c.   Since I wrote reference 1.a., I have learned that the area has been
subjected to ground vibration due to earthquake shaking. Enclosure 1, fur-
nished to me by IDNR, is a measurement of earthquake induced vibration in the
area. Reference 1.d. describes data from an earthquake that occurred in 1987
which produced peak particle velocities of 0.20, 0.22, 0.25, and 0.44 inches/
sec at instruments located in Daylight, IN. Also, Modified Mercalli Inten-
sity V[1] damage has been reported in at least one modern earthquake at Evans-
ville (personal communication with Dr. Ted Algermisson of the US Geological
Survey).

    d.   During a visit to the area on 15 Oct 91, I observed the construction
of two concrete block basements in progress (reference 1.e.). The block was
unfilled and unreinforced. Short dowels had been grouted into the top course
of blocks at intervals of about 10 ft which were to be connected to the sill
of the wood frame. Since these dowels extended only into the top course, any

---

    [1] "V.   Felt by nearly everyone; many awakened. Some dishes, windows and
other fragile items broken; a few instances of cracked plaster, unstable
objects overturned. Disturbance of trees, poles, and other tall objects
sometimes noticed. Pendulum clocks may stop."

2

CEWES-GV-A   9 Jan 93
SUBJECT: Addendum to 11 Mar 91 Memorandum for Record and Lateral Earth Pres-
sure Analyses for OSM Study

horizontal expansion or contraction of the superstructure due to temperature
changes would be transmitted to the top course of block and, if large enough,
could be a cause of the continuous horizontal cracks I observed in the mortar
just below the top course of block in many of the structures (see para. 8 of
reference 1.a.). The possibility of this type of construction detail did not
occur to me when writing reference 1.a. It, coupled with sufficient thermal
expansion, could be an alternate explanation for the horizontal cracking.

    e. Since I did not consider the possibility of earthquake induced ground
motions as a source for horizontal loading, and the combined effects of ther-
mal changes and the structural details to cause horizontal cracking, the
statement when I wrote reference 1.a.: "there are some types of cosmetic
damage present at some buildings that are clearly associated with horizontal
loading or movement and this author could find no source for such movement or
loading other than blasting" is no longer appropriate. I would delete it in
the light of better knowledge since I now can identify two possible other
sources for movements for consistent with the characteristics of damage that I
could not account for in Mar 91.

    f. The list of hypotheses in paragraph 5 of reference 1.a. should be
amended to add "(i) earthquake induced ground motion, (j) thermal cyclic
changes, and (k) frost action." Item (k) has not been discussed before (see
reference i). Where the foundation is above the frost line as in the case of
an exposed basement such as at the McCutchan residence, there is some risk of
frost or frost heave induced damage to the structure.

Earth Pressures

3. In paragraphs 9-16 of reference 1.a., Atterberg limit, X-ray diffraction
and consolidation test data on soil samples from the Daylight/McCutchanville
area were extensively discussed. These data indicated that some, but cer-
tainly not all, of the soils encountered in the top 10 ft were capable of
swelling upon the addition of water. Swelling pressures of 0.6, 0.6, 2.1, and
2.5 tons/sq ft were measured in four of the fourteen consolidation tests con-
ducted. No swelling occurred in the other ten. Atterberg limit data indi-
cated that expansive clay was present in 6 of 21 borings. In reference 1.a.,
I stated that "I could find no pattern...to explain why expansive clays are
present in some borings and not in others in the area." In reference 1.a., I
also noted that "there is an imperfect correlation...of bowed basement walls
with expansive soil. However, the Effinger residence, which has the most
seriously bowed-in basement wall, does not correlate."

4. In light of the concern over expansive clay, Interagency Agreement
No. EF68-IA91-13796 between WES and OSM (Section B.1.C.(1)) calls on WES to:
"(1) develop realistic bounding values for lateral earth pressures on basement

3

CEWES-GV-A   9 Jan 93
SUBJECT: Addendum to 11 Mar 91 Memorandum for Record and Lateral Earth Pres-
sure Analyses for OSM Study

walls, to include probably values for confined swell pressures in expansive
clays..." This section of this memorandum deals with this subject but will
not repeat material already included in reference 1.a.

5. As part of the recent multi-agency field investigation at Daylight/
McCutchanville, OSM contracted with commercial firms for (a) additional
borings in the vicinity of certain complainant and companion non-complainant
residences, and (b) for Atterberg Limits on selected samples from those
borings. A total of 85 samples from 21 borings were tested to determine
Atterberg Limits and/or gradation (see reference 1.g.). A total of 12 samples
from the additional borings had Atterberg Limits that would classify them as
being of medium of high swelling potential as shown in encl 2. It is inter-
esting to compare encl 2 to encl 2 of reference 1.a., a like plot of the
earlier data (which has been reproduced here as encl 3 for the reader's con-
venience). The only obvious difference is that a greater percent of the test
results fall below the A line. The gradation curves from the recent contrac-
tor performed tests suggest a sand size content that is unusually large for
what the geologists are describing as highly weathered shales. It may be that
the shale fragments were not processed correctly before conducting Atterberg
limits test in some cases and the presence of expansive clay is underestimated
by these data.

6. On 13-16 Oct 92 a meeting was held at the Indiana Geological Survey (IGS)
in Bloomington, Indiana. Those present are identified in encl 4. The purpose
of the meeting was to review all the soil property and geophysical data col-
lected in the vicinity of complainant and companion houses and obtain advice
of IGS staff on interpretation of all the data but especially the natural
gamma logs. Judgments on the soil profile and the presence of expansive clays
considering all the available data are given in encl 4. Enclosure 5 compares
older Atterberg limit data, new Atterberg limit data, the interpretation from
encl 3 and the presence or absence of bowed-in walls. The correlation between
bowed-in walls and expansive clay at depths shallower than the bottom of the
wall is not good. Everywhere there are bowed-in walls and soil data available
there is expansive clay present but it is generally too deep in the profile to
produce severe wall loads.

7. Earth pressure calculations for several cases are presented along with all
underlying assumptions in encl 6. Lateral loads per unit length on basement
walls embedded 5 ft in soil were estimated as follows:

4

SUBJECT: Addendum to 11 Mar 91 Memorandum for Record and Lateral Earth Pres-
sure Analyses for OSM Study

Case	Load lb/linear ft	Remarks
At rest earth pressure	750	
Active earth pressure	400	
Swelling	3,750	No swelling soils proven to along side basement wall.

6 Encls

PAUL F. HADALA
Assistant Director
Geotechnical Laboratory

CF:
Mr. Peter Michael - OSM
CEWES-SS-A (Mr. Chiarito)

5

(IN)DIANA DEPARTMENT OF NATURAL RESOURCES
AMAX AYRSHIRE
R. McCUTCHAN

well Earthquake

Date: 9/26/90   Time: 8:25   Triggers: 125 dB .05 In/Sec   Serial No: 69

Analysis Results

Channel:	Acoustic	Radial	Vertical	Transverse	Vector Sum
Peak Amplitude:	0.02 Mb 100dB	0.06 In/Sec	0.02 In/Sec	0.06 In/Sec	0.07 In/Sec
Frequency:	12.4 Hz	4.9 Hz	2.3 Hz	6.0 Hz	N/A

Seismogram

Data Scale: Acoustic = 0.10 Mb/Div   Seismic = 0.04 IPS/Div   Time Scale = 0.50 Sec/Div

Acoustic — 1.00 Mb

Radial — 0.53 IPS

Vertical — 0.52 IPS

Transverse — 0.53 IPS

Cal
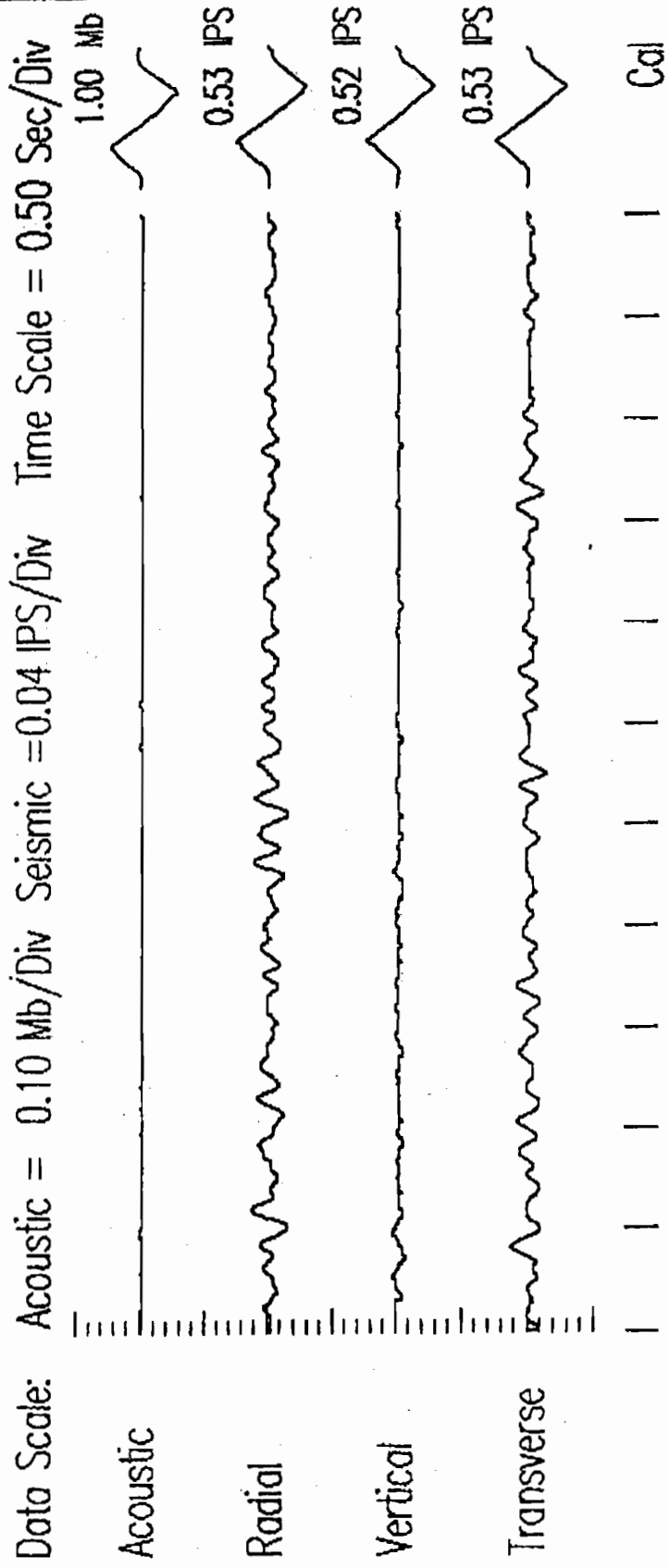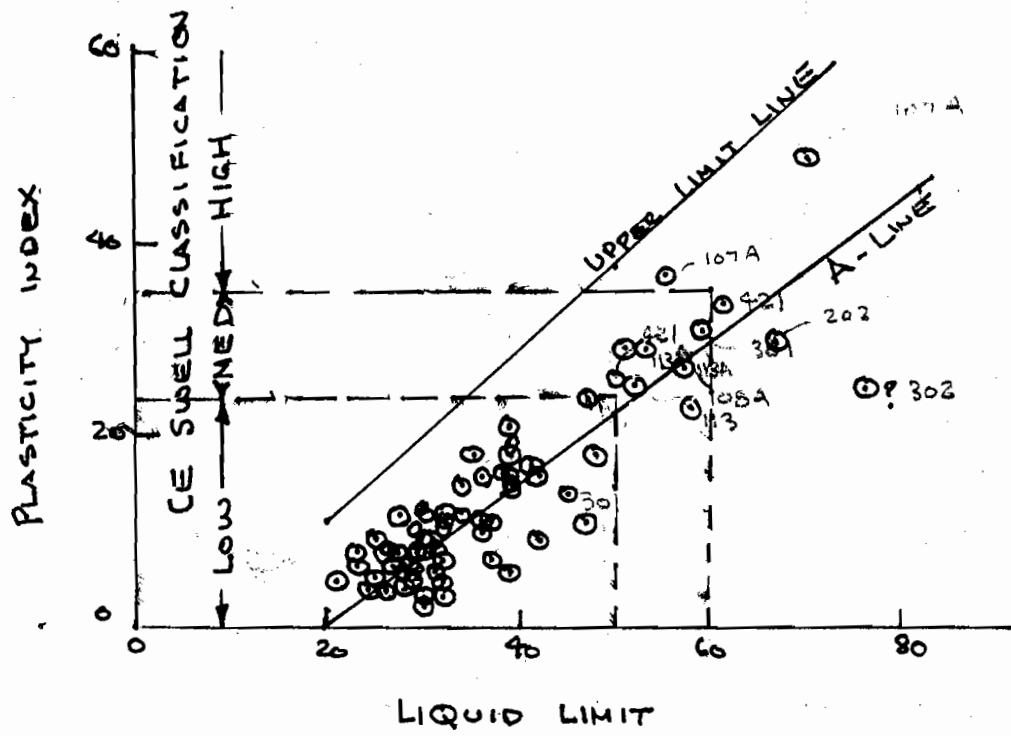
ATTERBERG LIMIT DATA

FINK 301 (2)
107A (3)(4)
ZINN 108A (4)
302 (4)
421 (4)(5)
202 (3)
113 (5)
113A (2)(3)(5)

Encl. 3

-71-
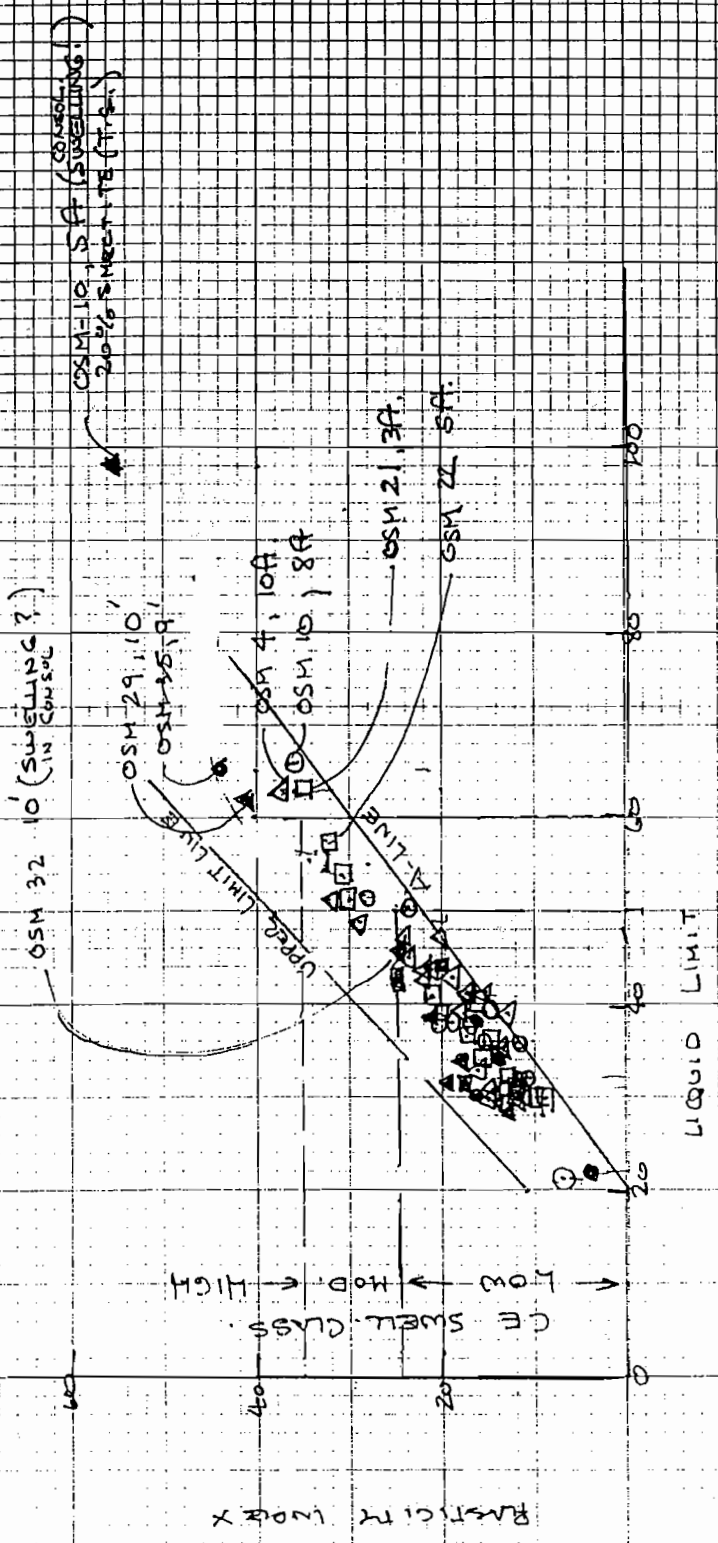
FOUNDATION/SUBSURFACE SUMMARIES FROM OCT. 13–16 MEETING IN BLOOMINGTON, ID.

· Interpretations by: Ned Bleuer and Don Eggert, Indiana Geological Survey, Paul F. Hadala, US Army Corps of Engineers, Bernard Maynard, Office of Surface Mining, Ken King, U.S.G.S.

NOTE: Bore holes at homes went to depth of hard drill resistance; at companion house to a depth of 9 foot. All depths are estimates.

ZIMMERMAN (Borings 103,)
10991 N. Green River Rd.

a. 0–7+–feet	–Dry, B horizon plus some loess.
b. 7–17 feet	–moist, silts and clays—no evidence of expansive clays from the samples.
c. 17–20 feet	–good compressional wave velocity change.
d. 17+– feet	–probable shale contact
e. 14'	–clay squeezed in the hole at this depth??
g. Foundation thought to be in loess.	
h. Comparison house is on very similar material in the upper 9 feet. (Shelton)(103A)	

CHRISTENSEN (Boring 115)(7)
Baseline Road

a. 0–7.5+–feet	–B horizon plus loess.
b. 7.5–10 feet	–Nonexpansive weathered shale?
c. 10 +– feet	–Bedrock ? sc/silt/sh interbedded (Shelburn Fm sp.?)
d. 8 and 11 feet	–good velocity shifts in compressional waves.
d. Subsurface formations are located in the Shelburn fm sequence below the West Franklin Ls.	
e. Foundation thought to be in nonexpansive weathered shale.	
f. Comparison house is on similar material in the upper 9 feet.(Klausmtach)(115 A)	

RICHEY (202)(35
15101 Cemetery Rd.

a. 0.5 feet	–B horizon plus clay/loess—nonexpansive.
b. 5–9 feet	–weathered shale ? –expansive material.
c. 9–10 feet	–weathered mix of materials, –colluvium?.
d. 10 feet	–West Franklin material (Ls, etc.)
e. 10 feet	–good compressional wave velocity change at 9–11'.
f. Foundation. The north foundation may be on loess and the south part of the foundation may be on the weathered shale.	
g. Basement wall is at same level as the expansive weathered shale.	
h. Comparison house bore hole samples do not show the weathered shale layer. The subsurface material goes from loess to colluvium to shale. Stevens (202A)	

OSBORNE (421)(29)
2400 Schlensker Rd.

a. 0–7 feet	–B horizon plus loess (reworked?) nonexpansive clays.
b. 7–8 feet	–Stoneline material; sd, lean clay, good permeability, good down slope (gravity) drainage. 13 blow count.
c. 9–12 feet	–weathered shale, moderate expansive (less than McCutchan) 26 blow count.
d. 12 feet	–competent shale.
e. 9 and 12 feet	–good compressional wave velocity changes.
f. Foundation is probably in the weathered shale layer.	
g.Comparison house bore hole samples showed no weathered shale, Stoneline is at 8 to 12, shale at 12'. (Rozanski)(421A)	

Encl 4

1

information handwritten add Dec 1992

BOETTCHER (ᴜᴌ₃ , ₃₃)
8261 Petersburg Rd.

 a. 0–5 feet   –B horizon and loess
 b. 5–8,9 feet   –Colluvium mix
 c. 8–9 feet   –weathered shale, <u>marginal expansive</u>
 d. 10 > feet   –competent shale.
 e. Compressional velocity change at 9+–feet.
 f. <u>Foundation</u> upper part in loess, lower on weathered shale.
 g. <u>Comparison house</u> foundation is on shale with slightly expansive weathered shale above. (Ogg)ᵢ(113 A)

HARRIS (ᴌᴏ7 , 4)
8304 Whetstone   ↑ Gore

 a. 0–7 feet   –B horizon and loess (<u>nonexpansive</u> clay and silt)
 b. 7–8 feet   –Stoneline colluvium. Probably <u>nonexpansive</u>.
 c. 8/9 feet and deeper   –Competent shale.
 d. 14 feet   –first significant velocity change in compression wave.
 e. 9 foot   –velocity change in shear wave velocity.
 f. No expansive clays at this location were found in the lab tests.—no tests in the 7–9 foot interval.
 g. Strange gamma log kick at 7 foot.
 h. <u>Foundation</u> thought to be on different material; that is, the center may be on competent shale where as each end may be located on the colluvium.
 i. <u>Comparison house</u> bore hole shows <u>expansive</u> ch clays at 8–12' depth which is probable below the foundation level. (Deutsch)(107 A)

GREENFIELD (₃₀₂)
8010 Petersburg Rd.

 a. 0–4 feet   –B horizon and loess.
 b. 4–6 feet   –weathered shale (no knowledge on expansiveness)
 c. 6–8 feet   –underclay, silty clay, <u>borderline</u> expansive.
 d. 8–10??feet   –silty clay
 e. 9.5–11.5 feet   –<u>very expansive</u> clay.
 f. >11.5 feet   –shale, siltstone, silty shale–unnamed Shelburn (sp) member just below the West Franklin.
 g. No significate compressional velocity changes in the upper 20 feet. Velocities show constant increase with depth.
 h. Many vertical (2–3+'deep) water drainage holes found down hill from house and lawn continued to shift level.
 i. Desiccant of underclay, (cracks) is possible mechanism for concentration of ground water flow and erosion of holes.
 j. <u>Foundation</u> is in/on underclay.
 k. <u>Comparison house</u> foundation is probably on shale at 8 foot depth (Palmer)(302 A)

2

FINK (301) ✓
9120 Old Petersburg Rd.

- a. 0–8.5 feet — −small B Horizon and loess
- b. 8.5–11 feet — −colluvium, high permeability
- c. 11–14 feet — −weathered shale.
- d. 14> feet — −shale, siltstone.
- e. A moderate compressional velocity change is located at approximately 8–9 feet and at 11–12 feet.
- f. No expansive soils were found by the lab tests.
- g. Foundation is thought to be in the loess.
- h. It is thought that the shale (14') is an aquatard, the colluvium is the possible "pipe" to carry loess away from the foundation if a source of water can be found.
- i. Conduct 301A is companion house.

EFFINGER (201)(325)

- a. 0–9 feet — −B horizon and loess
- b. 9–10.5 feet — −expansive material (it took 2.1 tons/sf to contain it??)
- c. 9–11 feet — −colluvium and expansive.
- d. 11–14 feet — −weathered shale
- e. >14 feet — −shale
- f. No large velocity changes in the compressional wave, a possible large velocity change at 9 feet in shear wave.
- f. Foundation is probably in loess and/or colluvium.
- g. It is possible that the foundation is on different materials as a 2nd bore hole further away from the foundation showed a greater thickness of weathered shale and less colluvium.

McCUTCHEN (108)(10)
9435 Baumbart Rd.

- a. 0–4 feet — −B horizon and loess
- b. 4–5 feet — −mixed colluvium, piping?
- c. 5–9 feet — −weathered shale, expansive– 2.5/sqft
- d. 10+–>feet — −firm shale
- e. Large velocity mismatch at 13 feet (compressional), 4 feet, 9 feet (shear wave).
- f. The colluvium may be the source of piping.
- g. The weathered shale shows slickensides (indicates movement at some time).
- h. Foundation is probably in weathered shale.
- i. One corner of foundation could be on firm shale (uphill side).
- j. Comparison house is Zinn (108 A)

ZINN (108 A)
9455 Baumgart Rd.

- a. 0–2 feet — −fill?
- b. 2–7 feet — −B horizon and loess
- c. 7–8 feet — −colluvium mix
- d. 8–10 feet — −weathered shale–v. expansive
- e. >10 feet — −shale
- f. A velocity change at 13–14 feet (compressional) a shear wave velocity change at 10–11 feet.
- g. Foundation is probable in the loess and above the expansive weathered shale.

3

Relationship of Data Indicative of Swell and Bowed-In Foundation Walls

Residence	No.	Lab Indication of Swell*				Interpretation from Meeting at IGS#	Basement Wall Remarks	Lab Test Data Remarks
		Old OSM Boring		New Boring				
		No.	Depth	No.	Depth			
Richie	202	35	9.5	202	7.5-9	5-9 expansive weathered shale	Bowed-in	0.6 TSF swell pressure measured at 10 ft
Osborne	421	29	9.4-10.0	421	7.5-14	9-12 expansive weathered shale	Bowed-in	
Boetcher	113	33	None	113	12.5-14	8-9 marginally expansive	--	
Ogg	113A	33	None	113A	5.0-14	4-6 slightly expansive	--	
Harris	107	4	10.1	107	None	None	Slightly bowed-in	
Deutch	107A	None	--	107A	7.5-11.5	8-12 expansive clay	--	
Greenfield	302	None	--	302	10.0-11.5	6-8 marginally exp., 9-11.5 very expansive	--	
Effinger	201	32	9.5-10.0	201	None	9-10.5 expansive	Severely bowed	2.1 TSF swell pressure measured at 10.0 ft
McCutchan	108	10	5.0-7.3	108	None	5-9 expansive weathered shale	--	2.5 TSF swell pressure measured at 5.0 ft
Zinn	108A	None	--	108A	10.0-11.5	8-10 expansive weathered shale	--	
Kinney	11.4	None	--	None	--	No interpretation made	Bowed-in	
Fink	301	None	--	301	5.0-6.5	None	--	
Residence not studied	20	20	14.6	None	--	No interpretation made	--	
No residence nearby	22	22	4.6	None	--	No interpretation made	--	0.6 TSF swell pressure measured at 10.0 ft
No residence nearby	17	17	None	None	--	No interpretation made	--	

* Atterberg limits indicating moderate or severe swelling by CE criteria.
# See encl 3. Judgment based on boring logs, lab data and natural gamma logs.

Encl 5

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Earth pressure at rest

$$K_o \approx 1 - \sin\phi$$

K_o = earth pressure at rest coef = $\dfrac{\sigma_H}{\sigma_V}$

σ_H = in situ horizontal stress

σ_V = in situ vertical stress = γz

γ = moist unit weight

z = depth

Active earth pressure

$$K_A = \frac{1}{N\phi} - \frac{2c}{\sigma_V}\frac{1}{\sqrt{N\phi}}$$

ϕ = apparent friction angle

C = apparent cohesion

H = embedded height of wall

K_A = active earth pressure coef. = $\dfrac{\sigma_{Ha}}{\sigma_V}$

Passive earth pressure

$$K_P = N\phi + \frac{2c}{\sigma_V}\sqrt{N\phi}$$

K_P = passive earth pressure coef = $\dfrac{\sigma_{H\phi}}{\sigma_V}$

$$N\phi = \tan^2(45 + \phi/2)$$

for $\phi = 30°$. $N\phi = 3$

for $\phi = 0°$ $N\phi = 1$

limit of active Rankine wedge

limit of passive Rankine wedge

Floor joist

A

concrete block wall

backfill

H undisturbed soil

Concrete slab B

pea gravel
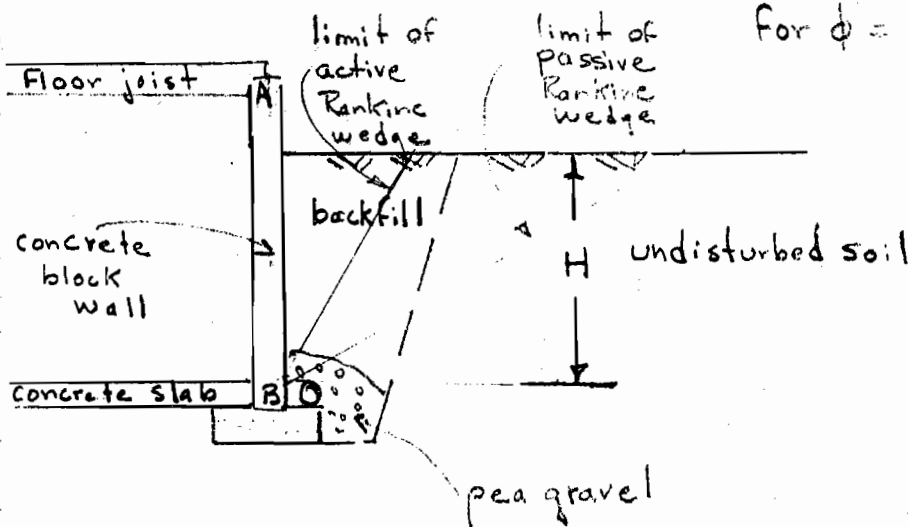
note that active wedge is mostly in backfill and that formation of passive wedge involves undisturbed soil.

There is absolutely no data on backfill

Encl 5

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Assume backfill identical in str. properties and density to undisturbed soil.

This will probably under estimate active pressure. and over-estimate passive pressure.

$$\int_0^H \sigma_H \, dz = P_0 = \text{total horiz. load on wall,}$$
at rest case

$$\int_0^H \sigma_{Ha} \, dz = P_A = \text{total horiz. load on wall,}$$
active case

$$\int_0^H \sigma_{Hp} \, dz = P_P = \text{total horiz. load on wall,}$$
passive case

At rest case

$\gamma = 100 \text{ lbs/cu ft}$ $\gamma_w = 62.5 \text{ lbs/cu ft.}$

$k_0 = 1 - \sin \phi$ $\phi = 30°$ for consol. drained state
$c = 0$

$k_0 = 1 - \sin 30° = 0.5$

$P_0 = \frac{1}{2} k_0 \gamma H^2 = \frac{1}{2} 0.5 \times 100 \times 25 \cong 750 \text{ lb/lf.}$

If drain became inoperable and the backfill became soaked

$$P_0 = \frac{1}{2} k_0 (\gamma - \gamma_w) H^2 + \frac{1}{2} \gamma_w H^2$$

$$P_0 = 0.25 (37.5) 25 + 62.5 \times \frac{25}{2} \cong 1000 \text{ lb/lf}$$

If backfill is well compacted (not likely in residential const., K_0 could be as high as 0.8 (page 100 ref h.).

Active case

(See sketch on page 1).

For this case to develop, point A or B must move inward a small amount. Wall friction is assumed zero. It is not but there are so many other unknowns in this estimate that the extra sophistication required to introduce it doesn't make sense.

$0.005H = 0.005 \times 5 \times 2 = 0.3$ in is an estimate of the "small amount"

drained str. case $\phi = 30$ $c = 0$ (See ref a for str. data)
H=5

$$\therefore P_A = \int_0^H \frac{1}{3} \gamma z \, dz = \frac{1}{3} \cdot \frac{1}{2} \gamma H^2 = \frac{1}{3} \cdot \frac{1}{2} \times 100 \times 25 \cong 400 \text{ lbs/lf.}$$

Active Case (Continued)

$\underline{\text{undrained strength case}}$ $\phi = 0$ $C = 1500 \text{ psf}$. See encl to ref c for strength data

$$P_A = \int_0^{H=5} \gamma z \left(1 - \frac{2c}{\gamma z}\right) dz = \int_0^5 100z - 3000 \, dz$$

$$= 100 \frac{z^2}{2} - 3000z \Big|_0^H = \frac{100 \times 25}{2} - 3000(5)$$

$P_A = 0$ neg result above indicate vertical cut 5 ft high can stand on its own for a short period

Passive Case

This case does not occur unless backfill is heavily over compacted or the backfill or natural soil swells. The passive case is the upper limit of swell induced load that can occur on an unyielding wall. (A or B do not move inward)

Swelling Case

3 Swelling Pressure Data Points Exist 0.6 TSF, 2.1 TSF, 2.6 TSF, 0 CT. (See reference a)

Arbitrarily swelling pressure cases of 2000 psf (1 TSF) and 4000 psf (2 TSF) will be examined.

It will be assumed that swelling occurs slowly and is resisted by drained passive earth pressure

$$\sigma_{H_P} = \sigma_v k_p = \gamma z \left(N\phi + \frac{2c \sqrt{N\phi}}{\gamma z} \right) \qquad \begin{array}{l} \phi = 30 \quad C = 0 \\ N\phi = 3 \end{array}$$

$$= 100z \left(3 + 0 \right) = 300z$$

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σ_H, horiz stress, PSF.

Depth below ground, ft.

assumed swell pressures
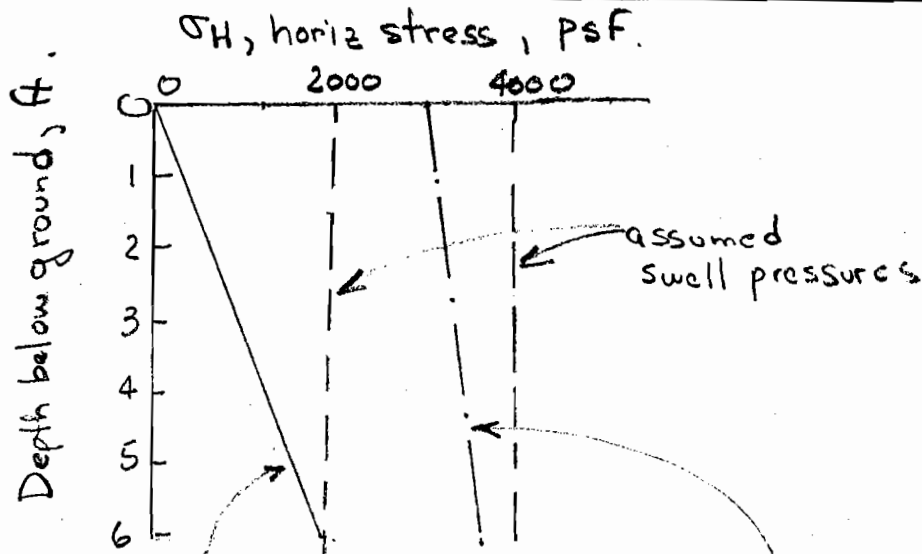
upper limit of long term pressure against unyielding wall.

$$P_P = \int_0^{H=5} \gamma z N\phi = \frac{\gamma \gamma z^2}{2} = \frac{3 \times 100 \times 25}{2} = 3750 \ \text{lb/lf.}$$

Suppose swelling occurred suddenly (unlikely), the upper limit is

$$\sigma_H = \gamma z \left(N\phi + \frac{2c\sqrt{N\phi}}{\gamma z} \right) \qquad N\phi = 1$$
$$C = 1500 \ \text{psf}$$

$$= 100 z + 3000$$

$$P_P = \int_0^5 100 z + 3000 \, dz = 500 + 15000 = 15,500 \ \text{lb/lf.}$$

I judge this an impossible case give the field conditions

Summary
1. Where swelling does not occur, total horiz load for H=5 ft is 400 - 750 lbs/lf.
2. Where swelling does occur and wall cannot move, the horizontal force on the wall is limited to 4000 lb/lf. or less.

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APPENDIX A

TRIP REPORT ON RECONNAISSANCE VISIT TO DAYLIGHT/MCCUTCHANVILLE

PREPARED BEFORE THIS INVESTIGATION BEGAN UNDER SEPARATE OSM FUNDING

(This memorandum is included because it is
referenced extensively in some chapters of this report
and is not generally available)

MEMORANDUM FOR RECORD

SUBJECT: Inspection of Building Damage Near Daylight and McCutchanville, Indiana, and Examination of Related Documents

1. Introduction. The undersigned, along with Mr. Vince Chiarito of the Structures Laboratory, WES, and respresentatives of the Office of Surface Mining (OSM), the Bureau of Mines (USBM), and the U.S. Geological Survey (USGS) visited 13 residences and two churches to the west of the Ayrshire Mine on 20 and 21 Feb 91 to observe damage to the buildings and to talk with the residents. I also reviewed all of the documents listed in encl 1. This memorandum will later refer to Reference numbers in this enclosure.

2. The OSM funded WES for this participation in the inspection team and the preparation of this memorandum under an Interagency Agreement dated 13 Jan 91. Approximately 100 property owners in the Daylight and McCutchanville areas have claimed that blasting by the mine has caused damage to their buildings, yet the distances between the buildings and the blasts and the charge weights involved in the blasts are within levels that, in prior U.S. blasting experience and literature of which this writer is aware, have not caused cosmetic or structural damage to buildings. Is this evidence of some heretofore unnoticed phenomena related to blasting or is the damage due to other causes?

3. The objective of my participation was to determine if the information available was sufficient to answer the above question and to render judgments as to possible causes of the damage.

4. Some key facts not in dispute. Damage complaints have been reported by only a small fraction of the building owners (about 7%). The damage is real in every case reported. The nature of the damage varies from cosmetic to structural. The vibration of the houses in response to ground shock and/or airblast is felt by and is audible to the area residents. Airblast itself is generally not audible. The mine opened in 1973. There have been a very large number of blasts over the years. Onset of significant complaints (November 1988) lags by a few months the change by the mine owner to cast blasting in March 1988. Cast blasting uses powder factors between 1- and 1-1/2 lbs/cu yd whereas the previously used method had typical powder factors of 3/4 lbs/cu yd. There are typically 3 shots/day every 3 days. Total charge ranges from 100,000 to 400,000 lbs and charges per delay range from 250 to 4,200 lbs. The March 1989 shortest distances from the highwall to Daylight and McCutchanville were, respectively, 9,500 ft. and 18,000 ft. The mine high wall has moved closer to these communities in the past few years, but will never reach them (under present local law) because surface mining is not permitted in Vanderburg County.

5. Hypotheses. Some hypotheses presented by various people in an attempt to explain the damage at various locations are:

a. Ground vibration of unusually high levels and/or unusually low frequencies caused by amplification due to topographic features and/or soil profile characteristics.

b. Airblast of unusually high levels due to unfavorable cloud cover, thermal gradients, wind directions, and/or topographic features.

c. Airport jet noise or sonic booms.

d. The presence of expansive clay minerals and cyclic moisture charges in the foundation soils is causing damage.

e. Liquefaction or pore pressure buildup under cyclic loading.

f. Subsurface erosion of soils near foundations due to lack of filters around underdrain.

g. Basement and/or footing construction practices which result in inherent weaknesses under static loads are responsible.

h. Combinations of some of the above.

6. Enclosure 2 is a portion of the Daylight and Evansville North USGS quadrangle maps showing the locations of the buildings visited, approximate boring locations, the mine and other relevant information. Enclosure 3 is a summary of notes prepared on the buildings visited.

7. Enclosure 3 is summarized in Enclosure 4. The most heavily damaged structures had significant differential settlement. Four of 13 structures with basements had bowed-in walls. The majority of the residences visited had surface water drainage and/or undrain inadequacies, some of which could be clearly linked to differential settlement patterns. This will be discussed in a later section. Expansive soil was present at 3 of the 15 structures and was not in evidence at 7 of 15. There is an imperfect correlation, in Enclosure 4, of bowed basement walls with expansive soil. However, the Effinger residence, which had the most seriously bowed-in basement wall, does not correlate.

8. The majority (8 of 15) of the buildings visited had pervasive fine horizontal cracks at or above ground line and/or other evidence of distress caused by an above or below ground horizontal loading. The closest building, St. John's, does not fall in this group, but all of the other buildings visited east of North Green River Road do (St. John's does not have a basement as the others do).

9. Expansive Soils. References 1 and 2 respectively contain Indiana Department of Natural Resources (IDNR) and Corps of Engineers (CE) water content, Atterberg Limits, shrinkage limits, gradation, Unified Soil Classification System classification data for more than 100 specimens collected from the study area which can all be used to infer swell potential using criteria given in Reference 5. Reference 2 also contains a few sets of consolidation test results that unequivocably determine the degree of swell potential. References 1 and 3 contain the results of X-ray diffraction analyses by others to determine the nature of the clay minerals present.

Some minerals, particularly smectite and morillonite, have extremely great swell potential if brought in contact with water.

10. The area has no past history of expansive clay soils. Enclosure 5 from Reference 4 and Figure 2-1 of Reference 5 both show no areas of expansive soil in the project area. Discussions and descriptions of the soil in Vanderburgh and Warrick Counties, contained in References 6 and 7 respectively also give no hint of the presence of expansive clays. Neither does Reference 8, a report on the bedrock in the area. Yet, as we will see, they are present.

11. The first piece of relevant data, a sample from the surface near house #108 (McCutchan) reported in Reference 9, was found to have 68% of its minus 2 μm fraction (25%) comprised of smectite and mixed layer smectite-illite minerals.

12. X-ray diffraction results by IDNR in Reference 1 are not consistent with those of the CE on samples from the same boring and depth range. IDNR indicates mixed layer clays, ranging from 24% to 61% of the minus 2 μm fraction and no smectite. CE reports one sample with 53% smectite, none with mixed layer minerals, and seven with vermiculite, a less expansive mineral but still one of concern.

13. In Enclosure 6, I organized all the soil laboratory data from References 1 and 2. Samples indicated by a dark dot • are clearly expansive by CE criteria and by the method of Van Der Merve (see Reference 5). All of the samples which are classifiable as expansive based on engineering properties and index properties have at least 10% by total weight of dry material of expansive clay mineral. However, many samples with more than 20% vermiculite clay mineral are clearly not expansive in flooded consolidation tests, and many samples with more than 20% of mixed-layer minerals do not have Atterberg limits suggestive of expansion. This inconsistency led me to relay heaviest on that which is easiest to measure and interpret in the laboratory and that which has the best empirical tie to past experience with expansive soils; i.e. the gradation, limits, water contents, and consolidation tests. I tended to discount X-ray diffraction results when they were inconsistent with the above. Enclosure 7 shows the Atterberg limits data from 3 ft or greater in depth plotted on a plasticity chart. Enclosure 8 shows PI vs % < 2 μm for the same data. Enclosure 7 shows six of the Atterberg limits classify the samples as medium swell potential and six classify as high out of a total of over 60 specimens. Twenty percent of the samples have some swell potential and three have marginal swell potential. Enclosure 8 indicated 13 samples with a high or very high swell potential by Van Der Merve's criteria. As indicated in the example in Enclosure 8, "very high" would produce a heave of 1.2 in. while "medium" would produce 0.3 in. heave in soil profiles like those seen at those few sites near Evansville where expansive clays were found.

14. I could find no pattern based on topography or geomorphology (Reference 10) to explain why expansive clays are present in some borings and not in others in this area. In general, the expansive clay, if present, was 3 ft thick or less and between 4 ft and 14 ft in depth. It was sometimes found just above weathered rock and sometimes in the middle of the soil column. All I can say with certainty is that in 6 of the 21 borings that had laboratory soil test specimens tested for limits and gradation, there was expansive clay

present, and in the others, there was not. Also where there were CE and IDNR soils lab data at approximately the same depth in the same boring, the two tended to agree.

15. About 28% of the borings had expansive clays present in a depth range where seasonal moisture changes could cause foundation movement. Enclosure 4 categorizes houses near certain borings as having expansive clay based on the data in Enclosure 6. Three of the four with bowed-in walls had expansive clay present. The borings were sometimes more than 100 ft from the house in question and the categorizations should be viewed with caution for this reason. Consolidation tests suggest swell pressures of 0.6 to 2.5 tons/sq ft at the 5- to 10-ft depth range in flooded consolidation tests. These loads would be sufficient to deform inward and severely crack unreinforced, unfilled concrete block basement walls of the types used in the houses visited. Still, the Effinger house which had severe wall damage does not have expansive clay in a boring made in the general area. I would like to see an undisturbed sample boring 5 ft from the north wall of the house with flooded consolidation tests on the samples obtained to determine whether there is a swelling soil near the house. Based on the information I have, I cannot say definitely one way or the other what caused the basement damage at the Effinger residence.

16. In addition to expansive soil, one must have a change in subsurface moisture to cause shrink swell phenomena. Enclosure 9 shows considerable variation in precipitation with seasonal and multi-year dry and wet spells. In the summer of 1988, a three-year low ended. Additionally, poor surface drainage control and partially plugged underdrains existed at some houses creating a situation where expansion could have been maximized coincidently with the start of cast blasting and heavy complaints. I conclude that expansive soil has contributed to the foundation damage at a _few_ of the houses, but by no means the majority. To say definitively which ones were affected and which were not would take a major separate geotechnical investigation at each residence.

17. Jet Noise and Sonic Booms. The property owners had not reported any sonic booms in the area or any damage due to aircraft noise. Four of the houses visited were fairly close to the Evansville Airport, but jet traffic there is light. The only indications of concern are found in Reference 9 which points out that a few of the complaints come from buildup "within 0.3 miles of the most active runway." The runway used for commercial passenger aircraft does not have a flight path over any of the houses we visited and is 3/4 mile from the nearest house visited. The references notes "upon working in some of the houses that aircraft operations caused structural rattling that could be both felt and heard." While aircraft noise is present, there is no mention of sonic booms anywhere in the record made available to me, and I feel that this subject should be dismissed from further consideration. Reference 9 notes that aircraft noise did not trigger instruments mounted in basements but that wall vibrations of less than 0.035 in/sec. were recorded at above ground wall and corner locations. These are insignificant levels and I recommend that aircraft noise be dismissed from further consideration. There simply is not any evidence that leads one to believe that a cause and effect relationship exists.

18. **Subsurface (Internal) Erosion.** The predominantly CL soils found in the area are moderately erodable and the underdrainage systems around the outside of the strip footings supporting the residences should be built as filters. Discussion with two homeowners and one builder who built his own home indicated that the common practice is to place 2 ft of pea gravel (or larger stone) around and over a perforated drain along the outside of the footing, cover this with a few inches of straw, and then backfill the area between the basement (or crawlplace) wall and the natural undisturbed soil with dumped material from the excavation.

19. To accomplish filtration, a layer of fine sand or a geotextile between the gravel and natural or backfill soil is required and apparently no one uses them in residential construction in this area. Examination of three underdrain outlet pipes at different residences indicated some degree of internal erosion. CL soils partically or totally filled the outlet. This material could be eroding from under the footings and, if it did erode, it would cause differential settlement. This is almost certainly what happened in the case of the garage floor at the Fink residence.

20. **Liquefaction or Pore Pressure Rise.** The soils at this site are not saturated. Initial saturation values reported in CE lab tests range from 56% to 98% with most below 90%. These soils will not liquefy when shaken severely. The Atterberg limits indicate that most of the soils present are also too plastic to be concerned with this issue. A key word literature search and the author's personal knowledge of the literature turned up no blasting or explosive testing experiences where such had occurred at vibration levels and scaled distances comparable to the Daylight-McCutchanville areas. Russian literature describes densification of wet loess with explosives, and U.S. literature describes densification of wet soils, but scaled distances involved are three orders of magnitude smaller than those present here. The question of pore pressure rise can be safely dismissed in my opinion.

21. **Settlement Under Repeated Low Level Vibration.** Noticeable vibration is occurring. In January 1988, there were 59 separate blasting events at the mine. In that year, there were over 500 events. Each event produces 10 to 20 cycles of motion (as shown by waveforms in Reference 9) in the frequency range from a few Hz to a few tens of Hz. This is 5,000 to 10,000 cycles per year, or perhaps 30,000 or more cycles in the period of interest. Vibration amplitudes are small, usually a few hundredths of an inch/sec based on Reference 9. What happens to this predominantly CL partially saturated soil with a void ratio in the 0.37 to 0.78 range when subjected to many cycles of low amplitude strain in a drained condition? I could find no data in the literature to answer that question and recommend that some drained long duration, torsion or axial vibration tests be run on drained, undisturbed samples from the site and that the sample be monitored for volume change. I also could not find any case reported in the literature where repeated vibration at the levels that appear to be occurring here caused differential settlements, so I have no reason to expect this to be a problem. However, I can't completely discount the possibility, hence I recommend a few tests.

22. **Surveys of Damage or Complaints.** We were briefed on one survey by OSM personnel which included only reviewes of 107 complaint cases. Of these, 6 had no damage, 62 had cosmetic damage, 32 had an intermediate level of damage, and 7 were seriously damaged. The number of complaints correlated temporaly

with the total charge weight used. The only other clear trend is that no two
story homes were rated as "severely damaged" (the Harris home is two story;
see my notes. Apparently the rating team did not call this severe).

23. Reference 9 describes a crack monitoring program conducted between
1 Nov 1989 and 3 Jan 1990. Forty-five cracks spread out over six residences
were examined 38 times for crack widening and/or elongation during the period.
Only one crack extension occurred and width changes noted were less than
± 0.1 mm. The latter was thought to be normal thermal cycling. Several
rather large blasts occurred during this period with a total of 59 blast
events. OSM displacement gages in the study houses also showed no changes.
This is strong evidence that blasting <u>as practiced at the time of the
Reference 9 study</u> was not causing damage in these six residences.

24. Reference 18 describes a survey conducted in 1977 of selected homeowners
out to six miles from the mine when the high wall was far to the east of its
present position. Complaints of damage decrease in number and severity with
distance. Beyond a distance of about five miles (within the glacial lake),
the trend of the least severe damage category reversed and actually increased
with distance.

25. <u>Airblast</u>. The author feels others in the review group have a better
basis for judgment in this area. Windows were not broken out. The
Christiansen home has windows with diagonal cracks, but this might have been
caused by differential settlement. Glass begins to break at 140 db, so we can
be confident that this level has not occurred. The airblast data in Reference
9 covers a short period of time and does not include all possible
meterological conditions which aggravate airblast or the largest events that
have occurred. The highest levels recorded in that reference was 121 db. OSM
criteria uses 134 db as a safety limit. There is no guarantee that airblast
larger than measured has not occurred in the Daylight and McCutchanville
areas. Reference 9 contains data which indicates that the structures which
were instrumented in the November 1989 to January 1990 time period responded
more to ground vibrations than to airblast.

26. <u>Ground Vibration</u>. Enclosure 10 is a summary of particle velocity
attenuation curves or data bands from several sources for studies near the
Ayrshire and Blanford Mines taken from References 9, 14, and 15. The scaled

ranges of interest exceed $8000 \text{ ft}/\sqrt{7000} = 95 \text{ ft/lb } 1/2$.   While the highest
particle, velocity recorded in the Reference 9 study was 0.1 in./sec,
enclosure 10 suggests that values of about 5 times this number could have
occurred at the minimum scaled range of interest in the northwest direction
from the mine in the vicinity of Daylight and values of 0.2 in./sec could have
occurred near McCuthanville. These are clearly large enough motions to
account for the perceptions of motion reported by the residents. However,
they are less than those at which damage is expected at frequencies of
vibration of 3 Hz or more. This is below the frequency range reported in
Reference 9.

27. The waveforms on pages 26 and 27 of Reference 9 suggest that most of the
structural response is the result of ground motion rather than airblast.
These waveforms do indicate that the predominant frequencies are 4 Hz or
higher.

28. I would like to see 5% damped pseudo-velocity spectra calculated for the actual radial and transverse wave forms obtained from the Reference 9 study (assuming original analog or digital time histories still exist) to see if lower frequencies than those which appear from visual examination of the waveforms can be found. I would also like to see ground motion data obtained with instruments with a 1/4 or 1/2 Hz lower limit of fidelity to see if there are any lower frequency components being missed.

29. Site Amplification. The topographic conditions at McCutchanville and the soft lakebed deposits do lend themselves to the amplification of ground motion as suggested in References 22 and 23. However, when the topographic and soil profile cross sections are plotted at common horizontal and vertical scales, the slopes appear rather mild and the amount of amplification to be expected is rather minor. This author has never encountered measured amplifications of 50 (as indicated in Reference 22) in experience with seismic ground motion amplification, and I don't know of a theoretical basis for values that large either. Five, rather than 50, might be a more reasonable upper bound. Regardless, the measurements made in Reference 9 and summarized in Enclosure 10 have already built into them whatever amplification factor nature gives the sites, so that issue is not really germaine to the question of how much motion occurred. What is germain and cannot be answered with the available data is the question: Were the source functions (the charges and delays) used before the period of USBM monitoring sufficiently different to produce higher amplitudes or lower frequencies of ground vibration?

30. Summary:

a. Expansive clay does exist at some locations where damage has been reported but does not exist at others.

b. There is imperfect correlation of expansive clay and bowed-in basement walls exists. Some of the damage seen is likely due to shrink-swell of expansive clays due to varying moisture conditions. The Effinger home is an apparent exception. Soils data should be collected nearer to the house if possible.

c. No credible evidence exists to support liquefaction, pore pressure rise, sonic booms, or jet noise as contributing to observed damage.

d. Some construction practices observed (unreinforced footings, unreinforced and unfilled concrete block basement walls, lack of filters around underdrain) and/or poor surface drainage control practice could aggravate or cause vertical differential settlement damage unrelated to blasting.

e. Although recorded levels of ground motion and airblast are smaller than established safety criteria, there are some types of cosmetic damage present at some buildings that are clearly associated with horizontal loading or horizontal movement, and this author can find no source for such movement or loading other than the blasting. There is no systemic pattern of these cases with distance or topography.

f. Unless there are lower frequency components present in the data from the Reference 9 study than those apparent on pages 26 and 27, the amplitudes recorded during that study should not have caused damage. Yet as indicated in e. above, there is damage likely due to vibration. It is possible that this damage occurred prior to November 1989 when other shot arrangements, delay combinations, and charge sizes were used.

g. If digital records exist of the ground motion data reported in Reference 9, 5% damped pseudo-velocity response spectra should be calculated from them to provide a better picture of their frequency content.
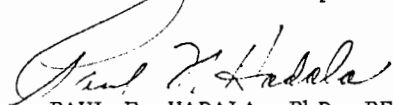
h. Measurements of broader band frequency response (i.e. 1/4 to 200 Hz), ground motion, airblast and structural movement in a 250,000 to 300,000 lb range total charge weight event at sites in the Daylight and in the McCutchanville communities are desirable to see whether there are any very low frequency components present that we are not seeing with the instrumentation used thus far.

i. We do not have data on the effects of long direction, very low amplitude sustained vibration on settlement of cohesive soils. A limited laboratory test program is recommended and will be proposed to OSM in a separate document.

j. Site amplification due to topographic effects and/or soft top layers is occurring at these sites, but its effects are already included in the Reference 9 measurements.

k. Since it is expected that there will be a continued effort to understand the wave propagation characteristics of this site, it is recommended that field cross-hole S-wave velocity measurements be made in the overburden soil and in the bedrock.

l. The bottom line of my profession judgment (and it is only a judgment!) is that blasting was responsible for some of the lighter damage seen in the Daylight and McCutchanville areas but that most of the damage that could be called major was due to swelling soil conditions, inadequate drainage, lack of filters, and/or unconservative foundation construction practices.

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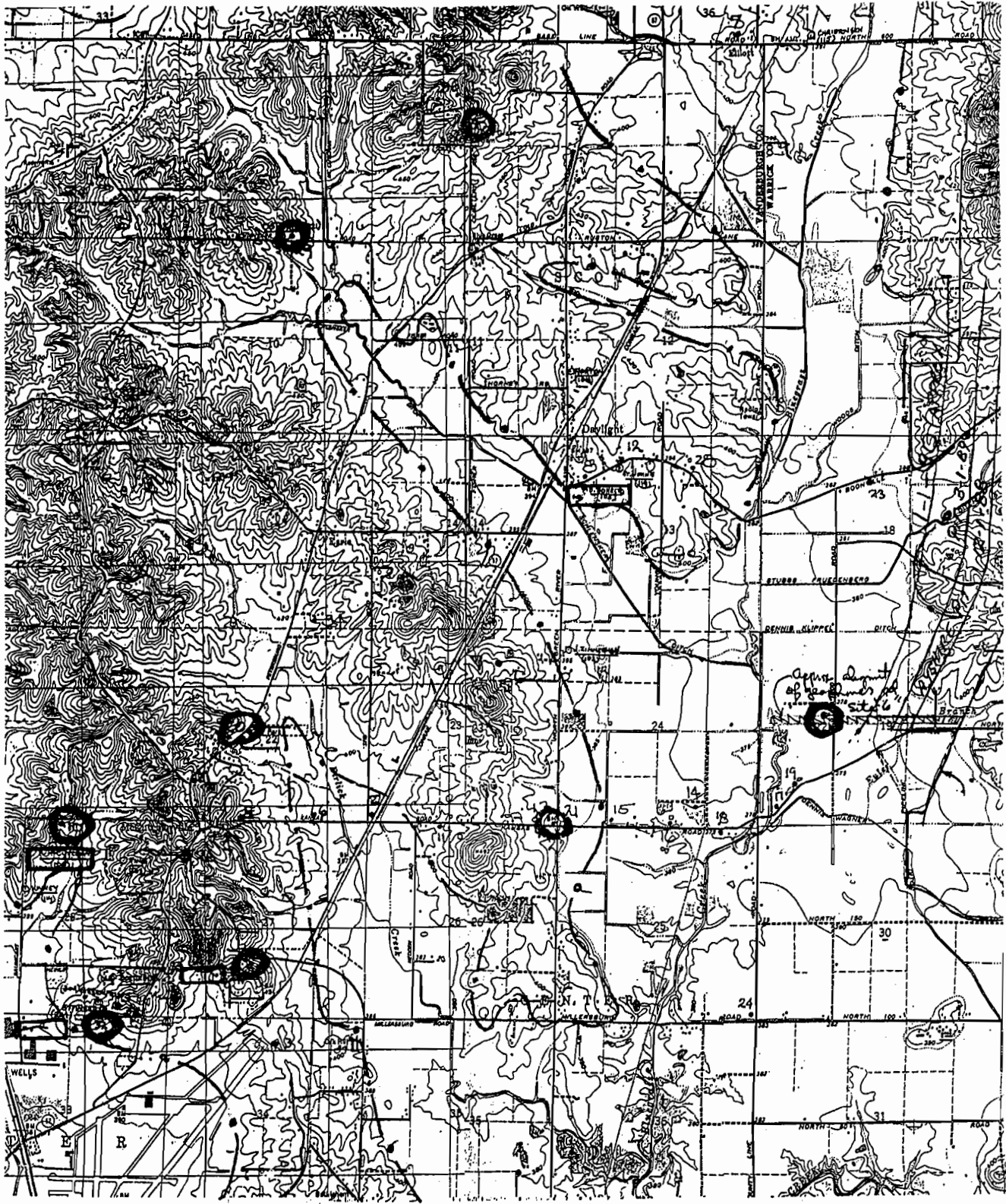
o Auger OSM ⎤ Source " Map Showing Thickness Contours of Unconsolidated Materials, OSM-McCuthanville Study Area
🖤 Rotary OSM ⎦

🖤 EXPANSIVE (˙) BORDERLINE EXPANSIVE

━━ Contour of 20' unconsolidated sediment.

▭ USBM/IDNR stations

SCALE |━━ 1 MILE ━━| -91-

NOTES ON OBSERVATIONS OF BUILDINGS VISITED

1. <u>McCutchan residence</u>.

   a. Side hill location with basement under all but garage. Unreinforced concrete block, basement walls on unreinforced footings. Long direction of the house E-W oriented toward the mine, water from roof collected in a cistern.

   b. Residents noticed plastic pipe break near basement wall, "shocks," "20-second vibrations," noise like "the pulling of a nail," "bump rather than sway," and a "thunder like" noise heard only once.

   c. Tile drain around edge of basement wall described by owner. There was no filter around the drain -- just large gravel. <u>Examination of the outlet showed ML-CL material which must have come from the vicinity of the building foundation</u>.

   d. The building is cracked structurally. <u>The basement floor is cracked N-S</u> and <u>there is a crack pattern in the outside wall consistent with the loss of foundation support on the downhill end of the building</u>. No evidence of foundation heave.

2. <u>Kinney residence</u>.

   a. Built in 1969. Concrete block, sidehill basement, one story.

   b. The W basement wall is cracked at mid height (below ground) with stair step corner cracks consistent with large lateral wall load below grade. The south basement wall is bowed in. There is a large crack running N-S through the basement floor. Superstructure interior cracking on the first floor consistent with basement floor crack.

   c. Posts on first porch were reported "hanging free" in March 1989. This is consistent with building trying to rotate downward in the downhill direction.

   d. Downspout drains had gaps where they should have contacted the subsurface portion of the drainage system.

3. <u>Greenfield residence</u>.

   a. This is a hill top (ridge top) residence with a basement. The owner said the house was built in 1961 and the cracks were not there in 1985.

   b. There are a number of interesting observations outside the building. The driveway has numerous parallel cracks that do not seem to be related to a pavement failure but a downhill movement to the N-W. There are small depressions aligned in a row in the N front yard downhill from the house. The

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topography outside the house has two "hollows; one to the NE of the house and one to the S. (Are there sinkholes in the area?)

c. The N basement wall has moved N and the structure has not moved with it. This is located where a conduit on the inside of the wall is pulled away from the wall. It is also evidents when standing on the east side of the house looking NW. The NE corners of the building do not align.

N

top of wall moved in or
bottom is moved out
reference to point A

A
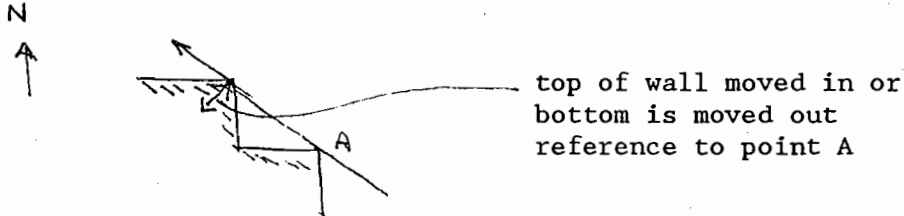
d. The east inside wall in daughter's bedroom has a diagonaly crack through the window consistent with base movement to the north.

e. The basement walls had horizontal cracks above the ground line.

f. Other interior damage observed were diagonal cracks in the living room running along the fireplace lintel, damage (molding pulled away) along the top of kitchen cabinets (the owner is certain this happened in the immediate time frame of felt vibrations).

g. The owner described the following; lampshades wiggle, pictures on the walls move. He does not hear any blast. He hears the house move. He can feel movement in his body and then hears the house rattle.

4. Fink residence.

a. This is a large older, well-built, two-story flemish brick house located on a ridge with a full basement. The owner spoke to us, but we were not permitted to go inside.

b. The garage floor (now basement) had major (>1 ft) differential settlement as a result of loss of support (i.e. voids). Outside near the sun porch on the S side of the building, there was a three-foot deep sink hole about 1-1/2 ft in diameter next to the basement wall.

c. The owner volunteered that gutters and drains had recently been installed to correct drainage problems and that the foundation undrain had been "roto-rootered" in January 1990. My own inspection of their outlets indicated partial filling with silty clay. Some (perhaps all) of the loss of ground being seem is probably due to lack of filter around the underdrain. The owner did not know how her underdrain was constructed.

d. Trees on the downslopes to the SW and SE of the house have a growth pattern indicating long term downslope creep.

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e. The owner said her basement was wet and muddy.

f. The outside walls were diagonally cracked near several windows.

5. St John's Roman Catholic Church.

a. This is a new church. This is the closest building to the mine we visited. It is constructed on grade.

b. The building vibrated about 11:15 a.m. while we were in it (a production blast occurred at about that time I later learned). There was no noise but a perceptable sensation of movement. It was not personally irritating.

c. The staff described some problems with windows and copper roof leaki in the church proper, but these had been corrected. No connection with blasting could be established.

d. The west outside block wall behind the altar had vertical cracks and there was a floor crack perpendicular to the outside wall nearby. There was 1/4 in. differential movement across the crack.

e. We observed cracked stained glass and a symmetrically cut frame in t daily mass chapel. This glass was removed from the old church and installed in the new chapel.

f. On the north and south sides of the long east wing, there were vertical cracks about 6 ft apart in the outside wall. There was a diagonal crack above the kitchen exterior door.

g. This is one of the least damaged of the buildings visited.

6. Christianson residence.

a. This is the northern-most home visited and is almost as close to the northern edge of the mine as St. John's is to the center. This property is either in or on the edge of the glacial age lake deposit, The terrain to th east is flat.

b. The house is a wood frame, permastone veneer, two-story structure wi a basement (that was dug after the house was built), and is one of the more severe structurally damaged homes visited.

c. Observations on the outside of the building indicated diagonal crack: on the east side under windows. There was a vertical crack in the chimney oi the west side.

d. In the interior, there were several cracks over the living room and master bedroom archway doors. The door to the glassed in porch and the porcl windows were all sticking. Porch window glass was cracked.

e. There were cracks in basement walls. (Field notes did not contain further elaboration.)

f.   The barn had unusual distress.   A roof support beam was town away from one of the wooden interior columns.

7.   Zimmerman residence.

a.   This is one of the closer residences and is also on the edge of the glacial lake.   It is a brick veneer one-story residence with a basement.   The house is 16-1/2 years old.

b.   Exterior observations included six horizontal cracks in the chimney, vertical cracks over the garage door.   Settlement of the front porch, settlement of backfill of basement excavation in rear side of house. Downspouts were separated from the below ground drains.

c.   There was a diagonal stair-step crack in the west basement block wall and basement floor cracks (without noticeable differential settlement).   There were continuous horizontal cracks in the basement walls above ground level and I could find no possible explanation for this except external horizontal loading applied from above or below.

e.   The owners' record of "nail pop" occurrence was interesting:

Date	Cumulative Number	
Jan 89	280	
Aug 89	397	
Sep 89	569	What kind of nails were
Jun 90	597	used to install sheetrock
Mar 90	636	was unknown
Feb 91	959	

f.   The owner described observations of chandeliers rattling and feeling a sense of movement in the basement floor as a result of mine blasts.

8.   Bohrer residence.

a.   This is a wood frame, brick veneer house.   There were interior horizontal cracks near the back door and diagonal cracks near windows.   There was a large diagonal exterior crack near a S side window and one near the garage door.

b.   There was a horizontal crack above ground level on the S side of the basement.   The sewer drain cracked at the junction of the cast iron pipe and the tile and others were cracked where the sewer drain exited the basement. This was one of the instrumented houses.

9.   Boettcher (Campbell) residence.

a.   This is an old L-shaped wood frame, brick veneer residence with a basement under part of the building and a crawl space under the rest.   The building is on a side of a hill.   The foundation walls are on a series of separate stepped footings which are not tied together.

b. The front exterior of the building had two large diagonal cracks, o1 in the NE corner that was aasociated with differential settlement of that corner of the building, and one that may have been caused by a tree root. (Large tree close to building in a location where there was no basement.) There were also diagonal cracks on the rear side of the building over the crawl space.

c. The basement floor was badly cracked. It was either center heave o1 edge drop. Based on the wall conditions, I am of the opinion it was edge drop. There were vertical and diagonal basement wall cracks. Exterior drainage of the building was recently repaired. I did not find any underdrை exits. I do not think this basement damage is vibration related. It is vertical rather than horizontal loading caused.

10. Effinger residence.

a. This is a one-story brick veneer home with a concrete block basement built by the owner in 1979. This residence is on the top of a ridge, relatively close to the airport and due west of the mine.

b. The owners described their feelings as "the house trembled" and "slamming from underneath." They did not feel any swaying movement of their bodies.

c. Outside the house I observed that the patio on the north side was tilted toward the house. I inspected the underdrain outlets. There was plenty of fall, but both outlets were partially full of ML-CL soil. The own said the drains were set on the footers, covered with 2 ft of pea gravel, a few inches of straw, and then dumped fill from the excavation. This does no satisfy filter criteria. The foundation excavation was only about 2 ft oustide the basement wall and was nearly vertical. It was dry during construction. The owner had a good set of construction photos.

d. The owner said the house was down 2 inches at the southwest corner (I had run levels) and he is sure that was not the way it was built.

e. There were horizontal cracks over the right (west) of the back door, also diagonal cracks on the NW corner. Horizontal and vertical cracks were under the windows on the N side.

f. There were four cracks in the R/C front port deck.

g. The N and E basement walls had heavy, open en-echelon diagonal and horizontal cracks.

h. The north basement wall was bowed in. The basement walls are about 9 ft high.

i. There was cracking in the SW corner of the basement room floor.

j. The owner said the basement floor in the NW corner became wet and muddy during rainstorms.

. Harris residence.

   a. This is a large two-story, wood frame, stone veneer building built in
53. It is oriented with the lag direction N-S and is on teh side of a
1ge. The owner moved in April 1988 and "felt vibrations fromthe start."
ᵊ whole house slammed, sometimes "from below" and sometimes "from above."

   b. The owner observed basement wall damage for the first time shortly
ter a felt event of November 16, 1988. The fall of 1988 was described by
s. Harris as "hideous" and she said that sporadic large events occurred
rough July 1990.

   c. Local damage around the top of pipe columns in the basement indicate
rizontal movement.

   d. One basement block wall was slightly bowed in.

   e. Large vertical cracks on both sides of the house, said to have been
iginally present in basement, were claimed to have been widened by recent
ents.

   f. Overall basement crack patterns suggest a N→S→N or vice versa
vement.

   g. The sun porch showed signs of outward and downward movement.

   h. On the first floor, there were diagonal or vertical cracks around most
ors and windows.

   i. The severity of damage in the basement was greater than the first
oor which was greater than the second.

   j. An air conditioner drain pipe break in the attic over the one-story
ırt of the building was consistent with the N-S movement observed elsewhere.

   k. This house leaves me with the impression that most of what I observed
ıs not the result of differential settlement or heaving. The pattern of
ımage was very consistent with overall racking of the house by horizontal
)ading.

2. Bluegrass Methodist Church.

   a. This is a two-story brick church with attached Sunday school classroom
ıilding built in 1964-65. These were the tallest concrete block walls
)served in our tour.

   b. The most prominent feature in the church sanctuary was a long,
orizontal crack about 3-4 ft above floor level in both sides which was said
o have occurred three years ago.

   c. Chandeliers on long chains in the church were said to rattle followed
y swaying.

d.   The most severely damaged room was a ladies rest room where the ?
school building connects with the church.

        e.   The entire crack pattern in the Sunday school building suggests
differential settlement of the N end, especially the NE corner.   Surface
, drainage near this corner was poor.   The damage in the church is unlikel)
be foundation related; most of the damage in the Sunday school could easi
caused by foundation settlement.

13.   Richie residence.

        a.   This is a hilltop residence NW of the mine founded above the
outcropping of the West Franklin limestone.   It is a single story stone \
with a partical basement.

        b.   The veneer had fallen off a portion of the west side of the house
The owner said this happened after a blast.

        c.   There were diagonal stair step cracks on N side consistent with N
corner settlement.

        d.   The owner stated he saw a crack in the ground surface form during
dry period after feeling the ground shake.

        e.   The owner said his cistern had cracked twice and had to be rebuil
twice.

        f.   The basement floor and west wall were heavily damaged.   The floor
cracked about 6 ft from the W outside wall.   The wall is bowed, visibly.
sketch below.

Basement

level here

2 in

|← 6' →|
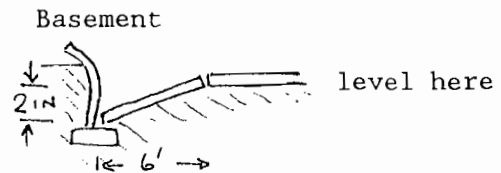
        g.   According to the owner, rock was at shallow depth under part of th
center.

        h.   The separate concrete block garage on a sidehill lcoation also
suffered diagonal cracking indicative of settlement of the downhill W foot
Drain pipe movement indicated rotation of the building sidewall in the E-W
plane.

14.   Osborne residence.

        a.   One-story, wood frame, stone veneer with crawl space built in 1955
current owner at a sidehill, uplevel location.   Basement added.

        b.   There has been settlement at the front of the house necessitating
front steps.   There is one stair step crack near the living room window.

c. On the rear (uphill) side of the house, there were several cracks, one which was wide.

d. There were stair step cracks on the east side of the house.

e. There were E-W ceiling cracks in front and back bedrooms.

f. There was a long horizontal crack at ground level in the basement 1.

g. The basement wall on the front side of the house was bowed in.

h. A separate concrete block garage had back wall stair step cracks bably due to differential settlement caused or aggravated by septic tank :fall. The ground was very wet.

Norton residence.

a. This is a one-story brick veneer and siding clad wood frame residence :h a basement.

b. The owner has lived in the residence since 1967. The house is older.

c. The carport floor has settled next to the house. There was a broken istic waterpipe under the carport that had to be repaired.

d. In the basement there was one floor crack in a large unjointed floor. ere was a horizontal crack at ground level in the basement that was tensive on the east side.

e. The garage had cracked floor slabs (it was a large pour with no nstruction joints). There was differential settlement in one corner that s attributable to poor drainage.

SUMMARY OF PERSONAL OBSERVATIONS

Structure	Long, horizontal Cracks at or Above Ground Level	Other Evidence of Horizontal Movement	Bowed Basement Walls	Differential Settlement	Slope Creep	Water or Drain Problems	Expansive Soils	Topography
McCutchan	No	No	No	Yes	No	Yes	Yes	x Side hill (M)
Kinney	No	No	Yes	Yes	No	Yes	- - -	Side hill
Greenfield	Yes	Maybe	No	Yes	Maybe	Maybe	No	x Hill top (M)
Fink	No	No	- - -	Yes	Yes	Yes	- - -	Y Ridge (M)
St. John's	No	No	No basement	Yes	No	No	No	Gentle slope (M)
Christianson	Maybe	Yes	No	Yes	No	- - -	No	⌐ Edge of glacial lake
Zimmerman	Yes	Yes	No	Yes	No	Yes	- - -	N Edge of glacial lake
Bohrer	Yes	Maybe	No	Yes	No	- - -	No	Gentle slope (M)
Boettcher (Campbell)	No	No	No	Yes	No	Maybe	No	✓ Side hill (M)
Effinger	See encl 3	Maybe	Yes	Yes	No	Yes	No	✓ Hilltop (M)
Harris	Yes	Yes	Slight	Yes	No	- - -	Yes	x Ridge top (M)
Bluegrass Church	Yes	No	No basement	Yes	No	Yes	- - -	
Richie	No	Maybe	Yes	Yes	Maybe	Yes	Yes	x Hill top (M)
Osborne	Yes	No	Yes	Yes	No	Yes	Yes	✓ Side hill (M)
Norton	Yes	No	No	Yes	No	Garage only	No	Gentle slope (M)

- - - = Don't know, (M) = Middle Geomorphic Surface

LEGEND

HIGH: HIGHLY EXPANSIVE AND/OR HIGH FREQUENCY OF OCCURENCE

LOW: GENERALLY OF LOW EXPANSIVE CHARACTER AND/OR LOW FREQUENCY OF OCCURRENCE

NONEXPANSIVE: THE OCCURRENCE OF EXPANSIVE MATERIALS EXTREMELY LIMITED

CATEGORY BOUNDARY

SOUTHERN LIMIT OF CONTINENTAL GLACIATION

NOTE: FOUR CATEGORIES OF EXPANSIVENESS ARE SHOWN ON THE DISTRIBUTION MAPS. THESE CATEGORIES ARE GENERALIZED AND QUALITATIVE AND ARE BASED UPON THE PRESENCE OF MONTMORILLONITE AND THE RELATIVE FREQUENCY OF OCCURRENCE OF ARGILLACEOUS MATERIAL IN THE AREA. MAJOR CATEGORIZATIONS ARE BASED UPON STRATIGRAPHY AND PHYSIOGRAPHY. DESCRIPTIONS OF THE PREDOMINANT GEOLOGIC FORMATIONS ARE GIVEN IN TABLE 1. THE BASIS FOR THE CATEGORIZATION IS EXPLAINED IN PARAGRAPH 22 OF THE TEXT.

Map compiled by D. M. Patrick, H. K. Woods, and Frederick L. Smith, Engineering Geology and Rock Mechanics Division, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Ms.
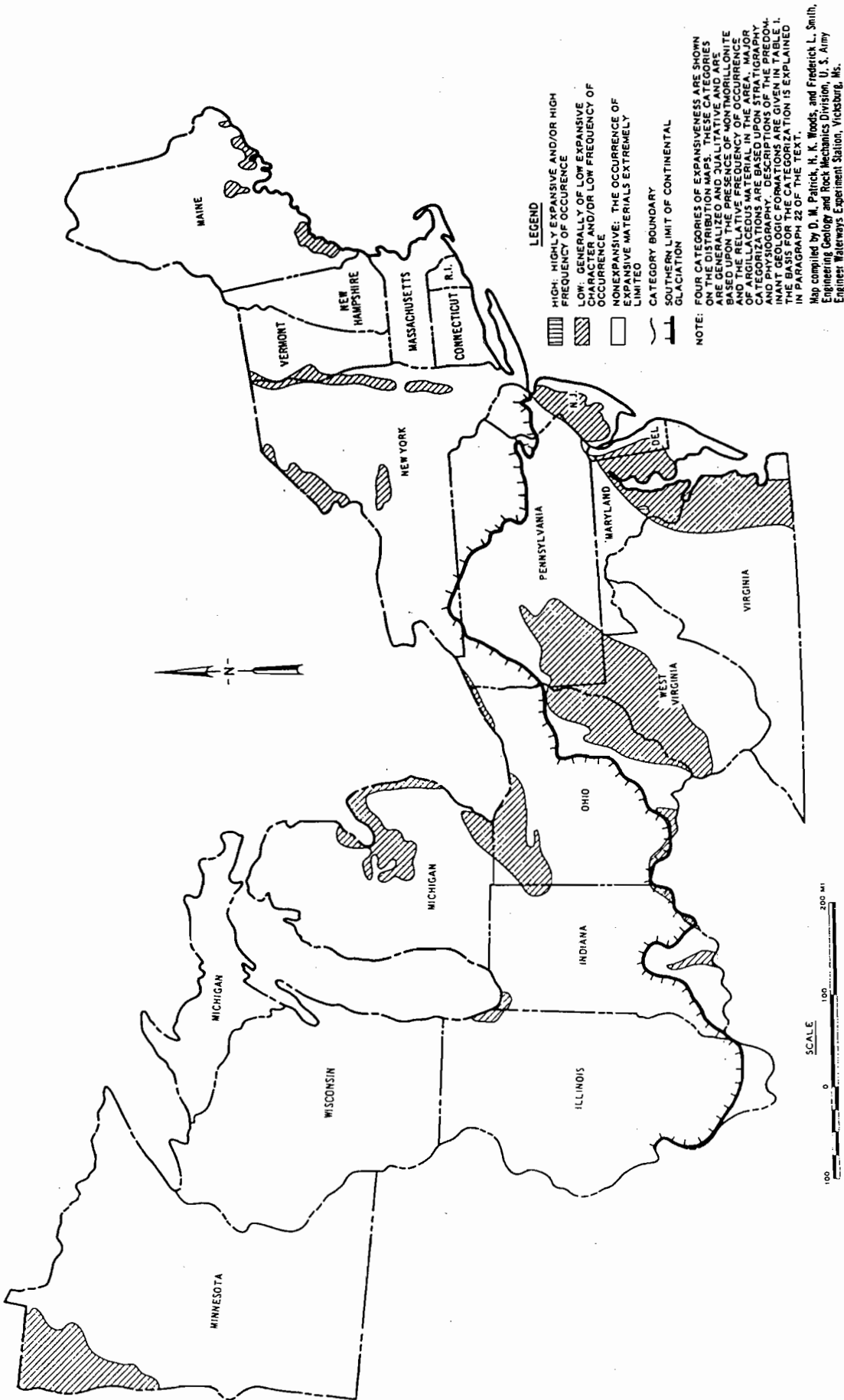
SCALE

100   0   100   200 MI

Figure 2.   Distribution of potentially expansive materials in the United States: FHWA Regions 1, 3 and 5

• = SWELL POTENTIAL

(•) = MARGINAL SWELL POTENTIAL

Geomorph Surf

* ⌈ MIXED LAYER
  | SMECTITE
  ⌊ VERMICULITE

Samples ≥ 4 ft deep , IGS Data

Boring	Depth	W/C	LL	PL	PI	%-200	%-2μ	SL	Smeared Shr
OSM-2(M)	4.5 56	—	36	21	15	98	13	24	5.8
	10.2	—	38	19	19	96	22	33(?)	2.3
	14.2 • 27	—	51	27	24	96	36	9	18.
OSM-3(M)	8.1 54	16-19	30	19	11	84	17	17	6.7
	12.3 43	17-20	40	22	18	95	27	18	9.8
OSM-4(M)	7.8 41	21-25	46	22	24	78	48	17	13.1
	10.1 • 61	20 ,	63	26	37	100	55	14	19.
OSM-5 (M)	4.2	∾14	31	16	15	70	19	11	10.
	9.7	16	34	17	17	71	25	13	10.2
	12.6 33 ✓	12-16	.44	23	21	75	27	18	11.9
OSM-8 (M)	7.3	16-17	31	19	12	95	17	17	7.3
	11.7 20	17.3	34	18	16	93	21	14	10.9
OSM-9 (L)	10.3	—	40	23	17	99	38	15	12.0
	13.0	—	37	22	15	99	38	—	—
OSM-10(M)	7.3 •	—	66	28	38	95	46	13	20
	11.1	—	39	24	15	86	18	17	10
OSM-12(M)	7.3	—	36	22	14	93	19	17	9
	12.8 13	—	32	19	13	96	22	14	9
OSM-13(M)	8.6	—	39	19	20	93	26	14	11
	12.4	—	31	18	13	93	22	18	18
OSM-19(L)	7.3	16-17	41	24	17	99	43	15	12
OSM-20(M)	7.8	—	32	16	16	82	22	13	9
	14.6 (•)	—	53	20	33	90	41	12	1
OSM-21 (L)	7.3	20-25	42	21	21	97	30	13	12
	(30-45)	(20±)	(63)	(27)	(36)	(99)	(51)	(8)	(28)
	15.1	18.6	30	21	9	96	20	—	—
OSM-22(L)	4.6 •	19.9	57	25	32	99	48	20	14

* % of -2μ Fraction ; No LABEL = MIXED LAYER   S = SMECTITE   V = VERMICULITE

Boring	Depth.ft	W/C	LL	PL	PI	%-200	%-2u	SL	%Lin Shrinkge
SM-29(M)	94•⁴⁵	—	75	2?	49	92	52	7	24.9
SM-32(M)	8.5	17.4	47	27	20	98	20	8	18.0
SM-33(U)	7.3	—	38	18	20	57	20	16	10.3
SM-34(M)	7.3	—	31	18	13	93	11	21(?)	14.3
SM-37(M)	6.5	—	31	21	10	97	23	24(?)	11.8

Samples ≥ 4 ft deep USACE Data

Boring	Depth.ft		W/C	LL	PL	PI	%-200	%-2u		
SM-1(U)	4.0	V=31	22.8	38	22	16	96	19	$e_0 = 0.72$	Consol, no swell
SM-4(M)	4.0	V=32	14.8	35	21	14	96	16	$e_0 = 0.67-0.59$	
SM-5(M)	4.0	V=37	16.5	34	16	18	66	18	$e_0 = 0.48-0.56$	Consol, no swell
SM-7(M)	4.0		22.2	35	21	14	96	13	$e_0 = 0.78-0.75$	
SM-8(M)	4.0		23.4	32	21	11	98	10	$e_0 = 0.69-0.75$	
SM-10(M)	50	S=53	21.0	98	25	73	81	37	$e_0 = .63-.80$	Consol swelling P=2.5TSF!
SM-12(M)	100	V=38	22.6	.39	19	20	96	24	$e_0 = .60-68$	Consol, no swell
SM-P-17(•)(L)	4.0		21.3	43	18	25	97	27	$e_0 = 67-64$	Consol, no swell
(•)	10.0		24.5	44	24	20	99	43	$e_0 = 66-0.70$	Consol swelling P=0.6TSF SL=17, LS=7%
	15.0		23.2	31	18	.13	96	15	$e_0 = 66-0.72$	Consol no swell
SM-29(M)	4.0	V=57	21.4	31	19	12	94	15	$e_0 = 63-69$	Consol no swell
•	10.0		24.1	62	21	41	93	49	$e_0 = 68-71$	Consol swelling P=0.6TSF
SM-30(U)	40		17.7	22	18	4	91	17	$e_0 = 60-67$	Consol, no swell SL=14 LS=8%
SM-32(M)	40		19.0	30	16	14	91	18	$e_0 = 0.58-63$	Consol, no swell SL=20 LS 11.2% P=2.1TSF
	10.0		13.7	47	23	24	99	30	$e_0 = 37-45$	Consol swelling
SM-33(U)	4.0	V=43	9.2	29	16	13	80	15	$e_0 = 42-77$	Consol, no swell. SL=14 LS=7.4%
SM-35(U)	4.0		17.2	30	15	15	78	20	$e_0 = 52-60$	Consol, no swell SL=11' LS=20.5%
•	9.5	V=48	18.2	65	21	44	99	49		

Swell 7
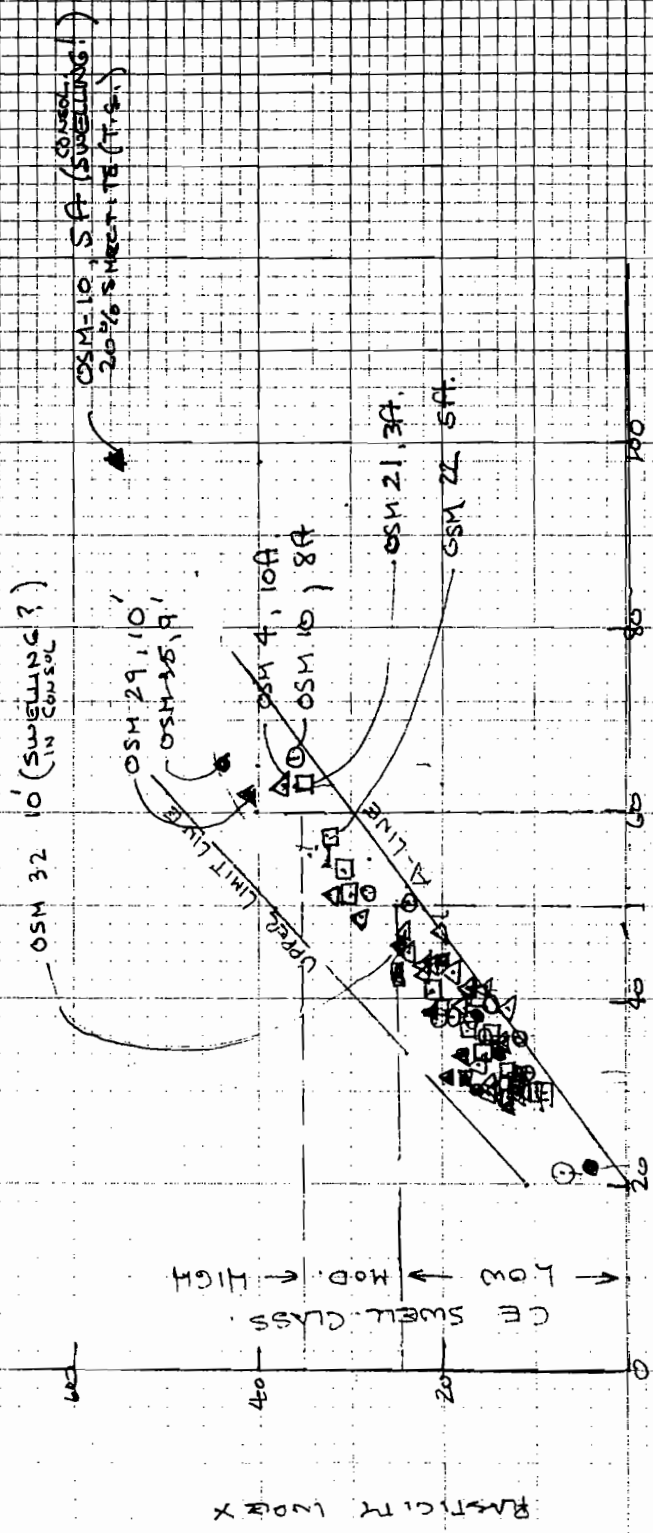
PLASTICITY CHART, DAYLIGHT & McCUTE

PLASTICITY INDEX

LIQUID LIMIT

CE	IDNR	Geomorph. Surf.
●	○	U	> 450 ft
▲	△	M	= 450 ft
■	□	L	< 450 ft

OSM-10, SA (consol-swelling!)
20% swell TE (T.S.)

OSM 32 10' (swelling? in consol)

OSM-29, 10'
OSM-25, 9'

OSM 4, 10'
OSM 10, 8'

OSM 21, 3'
OSM 22, SA

Upper limit line

A-LINE

CE SWELL CLASS
LOW → ← HIGH
HIGH ← MOD. → LOW

0   20   40   60   80   100

CLASSIFICATION OF SWELL POTENTIAL

10f

AMAX starts cast blasting

October 1984

December 1983

October 1981

October 1980

INCHES OF PRECIPITATION
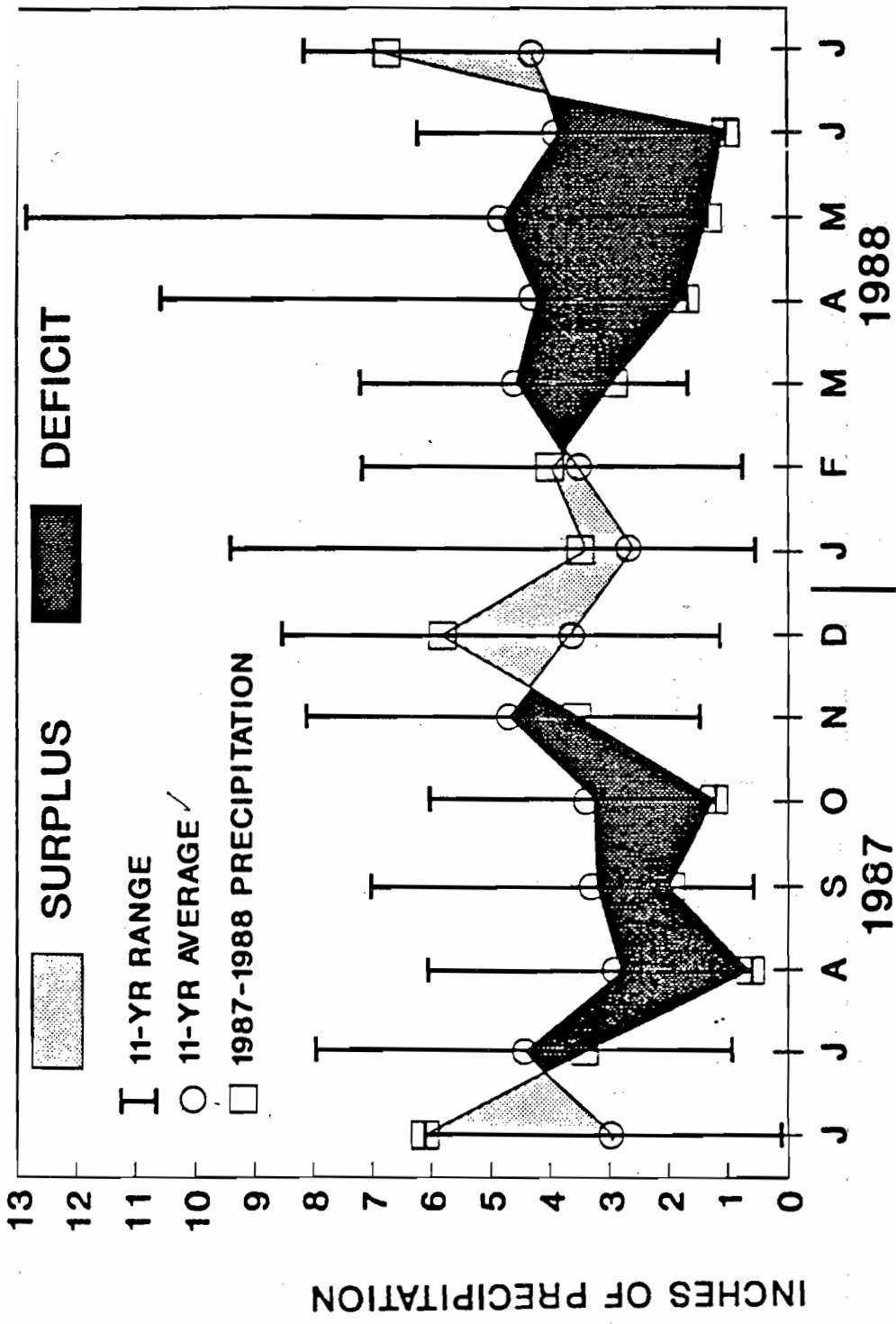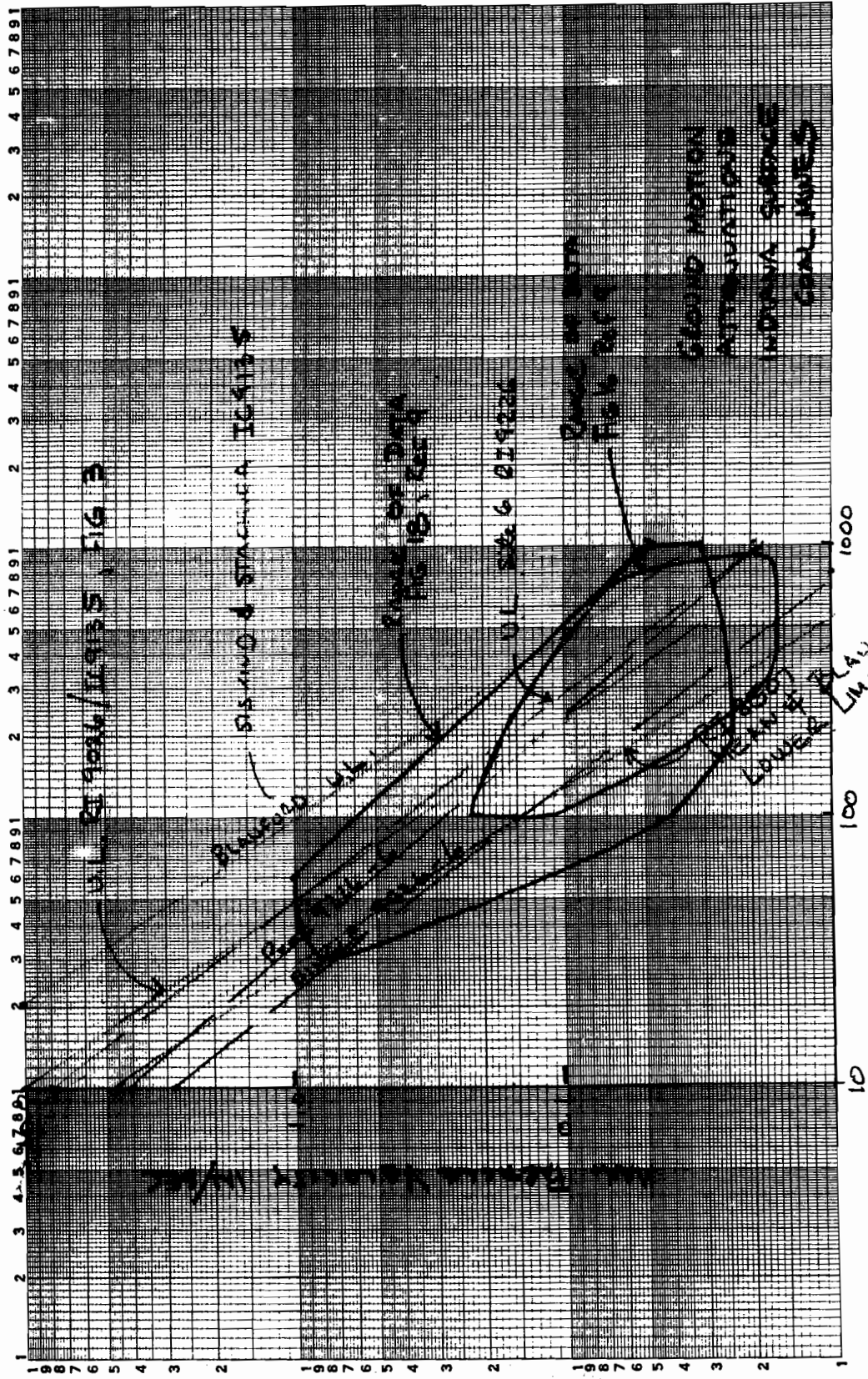
Fig. 9.

Figure 9.6-- Monthly Precipitation for Evansville, Indiana from June 1987 through July 1989.

SCALED RANGE $Ft/Lb^{1/2}$

Encl 10

APPENDIX B

ADDENDUM OF 30 OCT 93 TELEPHONE REQUEST

FROM SPONSOR

PART A

Bearing Capacity and Settlement Calculation for a Two-Story Residence

In October 1993, it was requested that a bearing capacity and settlement analysis parallel to that in Chapter 3 be performed for a bearing pressure o 3.15 kips/sq ft on a 20-in. wide strip footing. Using the chart on page 57, shear strength of 1.1 kips/sq ft is required for a factor of safety of 3.0 bearing capacity. The unconfined compression test data indicate this shear strength is exceeded in 12 of 15 cases. The standard penetration test data indicate this strength is exceeded in 58 of 63 cases. Bearing capacity is considered adequate for this case.

Settlements calculated for this same loading were as follows:

	Center Line in	Corner in
Consolidation	0.230	0.130
Elastic	0.045	0.045
Total	0.275	0.175

If rounded to the nearest 1/16 in., the text statements on page 61 and i paragraph 12 on page 55 are also valid for a 3.15 kip/sq ft bearing pressure.

PART B

Basis for "yes" and "maybe" entries in the third column

"other evidence of horizontal movement" on page 100

Mr. Peter Michael of OSM asked for this clarification in October 1993.
Response was as follows:

Greenfield: see items b and c, page 92-93.

Christianson: see item f, page 95.

Zimmerman: see items c and f, page 95.

Bohrer: see item b, page 95.

Effinger: see item b, page 96.

Harris: see items c, d, g, and j, page 97.

Richie: see item e, page 98.