International Conference on Case Histories in Geotechnical Engineering

(1998) - Fourth International Conference on Case Histories in Geotechnical Engineering

11 Mar 1998, 1:30 pm - 4:00 pm

Vibration Mitigation for Brewery Stockhouse Demolition

William J. Prosser
Woodward-Clyde International - Americas, St. Louis, Missouri

Thomas J. Mottin
Woodward-Clyde International - Americas, St. Louis, Missouri

George M. S. Manyando
Woodward-Clyde International - Americas, St. Louis, Missouri

Gary M. Andruska
Anheuser-Busch Companies, Inc., St. Louis, Missouri

Proceedings: Fourth International Conference on Case Histories in Geotechnical Engineering, St. Louis, Missouri, March 9–12, 1998.

Vibration Mitigation for Brewery Stockhouse Demolition

Paper No. 4-04

William J. Prosser
Associate, Woodward-Clyde International-Americas
St. Louis, Missouri USA

George M.S. Manyando
Project Engineer, Woodward-Clyde International-Americas
St. Louis, Missouri USA

Thomas J. Mottin (deceased)
Assistant Project Engineer, Woodward-Clyde International-Americas
St. Louis, Missouri USA

Gary M. Andruska
Engineering Manager, Anheuser-Busch Companies, Inc.
St. Louis, Missouri USA

ABSTRACT

Nineteen thirty's vintage reinforced concrete brewery stockhouses, collectively known as Borsari Cellars, were demolished to make space for the construction of a new stockhouse. (A brewery stockhouse is a refrigerated building containing beer storage or aging tanks.) The stockhouses to be demolished shared three common walls with two other stockhouses which were to remain intact during the demolition. It was necessary that the three shared walls remain attached to the remaining stockhouses and that the demolition take place without causing vibration damage to glass-lined tanks in the remaining stockhouse, adjacent stockhouses, and to several underground tunnels present below the demolition site. The following tasks were performed to successfully complete this project: (1) design and install a rock-anchored tie-back system for retaining the three shared walls; (2) evaluate ambient ground vibrations during normal business activities in the subject stockhouses and general project area; (3) recommend an allowable demolition vibration criteria and develop a monitoring program; and (4) implement the monitoring program. A resultant peak particle velocity (RPPV) of 1.0 inch per second was recommended as the threshold for low-risk demolition. This program was used successfully to demolish the Borsari Cellars without causing damage to adjacent stockhouses, glass-lined beer tanks, and underground tunnels on the project site. This approach could be used for similar situations or for demolition in areas where industrial buildings with sensitive equipment are in close proximity.

KEYWORDS

Ambient Vibrations
Demolition Vibrations
Resultant Peak Particle Velocity (RPPV)
Rock Anchor Tie-Backs
Vibration Monitoring
Demolition Vibration Spectrum

INTRODUCTION

A modernization plan for the St. Louis Brewery included the demolition of 1930s vintage, Stockhouses 10, 11 and 15 (Borsari Cellars), to create space for the construction of new Stockhouse 19. The Borsari Cellars shared three common walls with other stockhouses that were to remain. The location and layout of the Borsari Cellars in relation to other structures before demolition is shown in Figure 1.

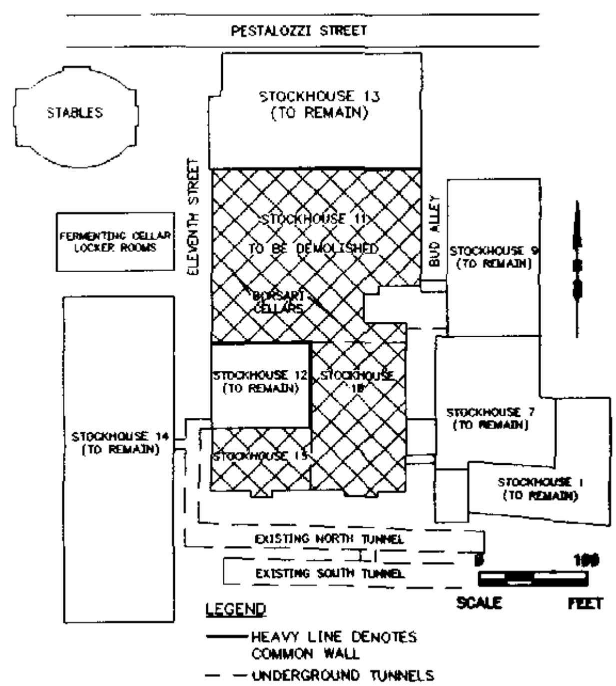

Figure 1: Site Plan before Demolition

The objective of this service was to provide a demolition vibration evaluation and monitoring program which would reduce the risk of vibration-induced damage to glass-lined beer tanks in adjacent stockhouses during the demolition of the Borsari Cellars. The Owner was especially concerned about maintaining the integrity of: 1) glass-lined beer tanks in Stockhouses 12, 13 and 14 which are typically supported by stirrups connected to the steel frames of the building; 2) several subterranean, utility-carrying, brick-arch tunnels; and 3) the north and east walls of Stockhouse 12 and the south wall of Stockhouse 13. These walls are the original exterior walls of the pre-existing Borsari Cellars. A system to maintain the stability of the shared walls of Stockhouses 12 and 13 was designed and implemented before demolition would begin. This paper presents the procedures followed to successfully complete this project.

The Borsari Cellars

A Borsari structure consisted of clustered reinforced-concrete frames which uniquely constitute the individual beer tanks. A typical beer tank is about 8 to 10 ft high, 15 ft wide, and 50 to 60 ft long. Five to six stories of the beer tanks, typically five to six tanks wide, comprise a Borsari Cellar. The plan dimensions of the cellars was about 200 by 300 ft. The Borsari Cellars were supported on footing foundations bearing on limestone bedrock. In most cases, the basement floors were also founded on bedrock. Stockhouses 1, 7, and 9 are also supported on footings bearing on limestone bedrock. Stockhouses 12, 13, and 14 are supported by drilled shafts socketed into limestone bedrock. The lower floors in these buildings are generally above the bedrock level. A typical cross section of a Borsari cellar is shown in Figure 2.

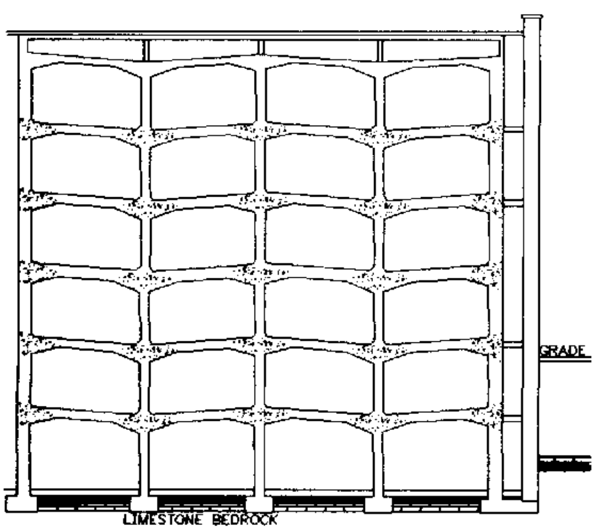

Figure 2: Cross Section of Typical Borsari Cellar

SCOPE OF WORK

This project consisted of three phases. Phase 1 consisted of two parts: A) Evaluating predemolition ambient ground and structural vibration levels resulting from daily routine business operations, and B) evaluating and implementing a stabilizing system for the shared common walls of Stockhouses 12 and 13. Phase 2 consisted of developing the allowable vibration criteria and the monitoring program, and Phase 3 consisted of demolition and implementation of selected vibration criteria and monitoring program.

PHASE 1: AMBIENT VIBRATION MONITORING AND COMMON WALL ROCK-ANCHOR TIE-BACK STABILIZATION SYSTEM

Ambient Vibration Monitoring

Typical ambient floor and structural vibrations resulting from routine business activities around the subject stockhouses were measured at 102 discrete locations established on structural members and on beer tanks using a Safeguard Seismic Unit, SSU 1000D. A SSU is a Microprocessor-Based Digitizing Seismograph manufactured by Geosonics Inc. The SSU 1000D measures peak particle velocities, PPV, along three orthogonal directions, (longitudinal, transverse, and vertical) and the corresponding frequencies at which the peaks occurred and then computes the peak particle displacements, PPD, and the peak particle accelerations, PPA. The SSU also computes the resultant peak particle velocity, RPPV, or the true vector sum. Phase 1 vibration monitoring results are presented in Table 1.

Table 1: Ranges of Amplitudes of Ambient Vibrations

Location of Vibration Monitoring		Vibration Parameter				
		PPV (mips)	PPD (mils)	PPA (%g)	Freq (Hz)	RPPV (mips)
Stockhouse 12	Structure	0 - 20	0 - 0.02	0 - 2.8	0 - 57	0 - 20
	Tank					
Stockhouse 13	Structure	0 - 50	0.01 - 0.25	0 - 4.1	20 - 57	0 - 50
	Tank	0 - 80	0 - 3.45	0 - 7	2.5 - 85	0 - 80
Stockhouse 7	Structure	0 - 20	0.02 - 0.23	0 - 2.8	20 - 57	0 - 20
	Tank	20 - 60	0.02 - 0.58	2 - 5	19 - 43	30 - 60
Stockhouse 9	Structure	0 - 20	0 - 0.28	0 - 3.5	10 - 57	10 - 30
	Tank	0 - 60	0 - 0.12	0 - 10	28 - 85	20 - 70
Stockhouse 14	Structure	0 - 20	0 - 0.14	0 - 3.5	20 - 57	0 - 30
	Tank	0 - 10	0 - 0.02	0 - 1.4	0 - 85	0 - 10

Note: 1.0 mips = 0.001 inch per sec
1.0 mils = 0.001 inch

These data were referenced during the Borsari Cellars demolition. A 24-hour continuous vibration monitoring was also performed on two individual beer tanks in Stockhouses 13 and 14 to measure vibrations during a tank cleaning operation. The results of these measurements indicated that PPV values approach 2 ips during tank cleaning, a process that consists of removing and replacing the beechwood chips used in the beer aging process. This relatively high value was attributed to the flexibility of the tank supports.

Common Wall Rock-Anchor Tie-Back Stabilizing System

To maintain the integrity of the remaining common walls at the north and east sides of Stockhouse 12 and the south side of Stockhouse 13, a stabilizing support system was selected and implemented. The common walls are about 2 ft thick and consist of a basal reinforced concrete wall supporting a non-reinforced brick masonry wall reaching a total height of about 60 ft. An internal steel frame supports the individual horizontal glass-lined tanks. Within the superstructure levels, holes were drilled into common walls and anchor bars inserted. The anchor head was inside the Borsari Cellar wall; the opposite side was attached to a structural member of the remaining stockhouse. A schematic of the connection of the Borsari Cellar wall with the remaining stockhouse is shown in Figure 3.

In basement areas, where significant lateral loads on the reinforced concrete walls were present because of the wall backfill and the floor loadings, a tie-back or rock anchor approach was required. A key factor which made the rock anchor approach attractive was the proximity of sound limestone bedrock to the basement floor and walls.

Rock Anchor Installation

A low head room drilling rig, positioned in a beer tank, was used to drill anchor holes into rock at an angle of 30° to 45° with the horizontal. Cable tendons and anchors were designed based on a bond stress of 100 psi. Anchor design loads ranged from about 2 kips to 100 kips, bond lengths ranged from 10 to 20 ft, stressing lengths ranged from 10 to 44 ft, and total anchor lengths ranged from 21 to 64 ft. Twenty-two rock anchors were installed in the north wall of Stockhouse 12, 37 rock anchors were installed in the east wall of Stockhouse 12, and 12 rock anchors were installed in the south wall of Stockhouse 13, for a total of 71 anchors. A schematic of a rock anchor installation is shown in Figure 4. Rock anchor installation was completed in accordance with specifications prepared by the Owner during the period of October 8, 1992 to May 10, 1993.

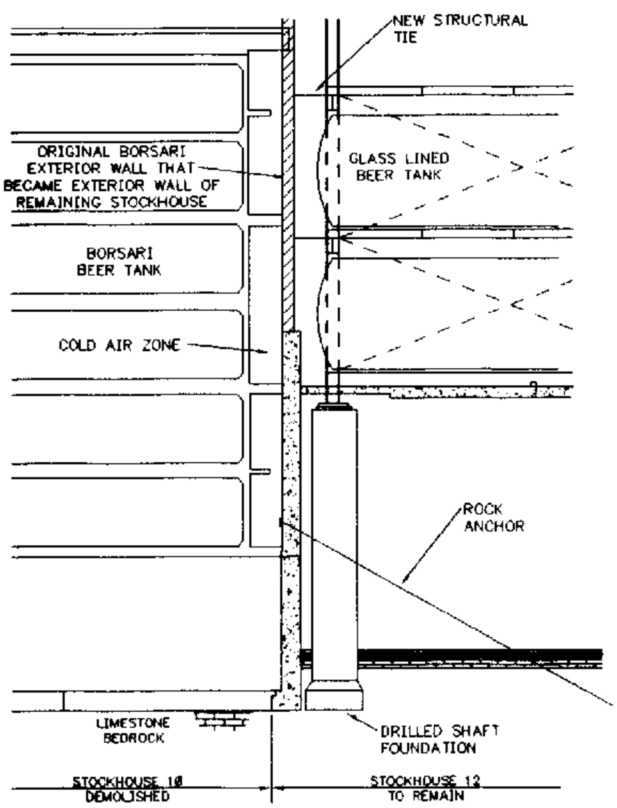

Figure 3: Schematic of Wall Connection

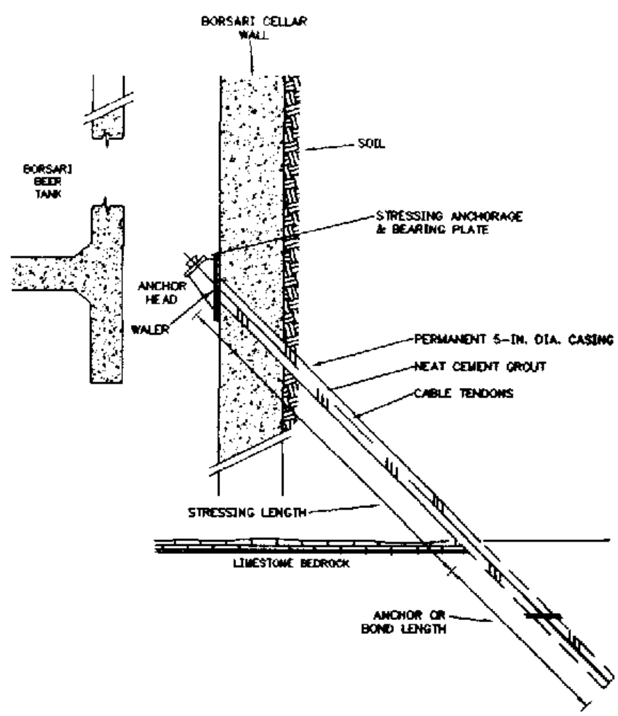

Figure 4 : Schematic of Rock Anchor and Tendon

The rock drilling was done using a 4-inch diameter carballoy button bit to the required depth into limestone bedrock. The hole was cleaned by either blowing with compressed air or flushing with water. The anchor hole was then filled with neat cement grout with a 1:1 water cement ratio by volume. Grout was introduced at the bottom of the hole using a grout tube to displace the water. The anchor was then inserted into the hole displacing the grout. Grout cube samples were taken from every batch of grout for compressive strength testing. The specified 28-day strength was 4,000 psi. The compression strength tests for all grout cubes exceeded 4,000 psi.

Double corrosion-protected Dywidag cable tendons were used as the anchors. The bond sections were 2-in. diameter tendons ranging in length from 10 to 20 ft. A bond design strength of 100 psi was used. Depending upon the design load, the stressing length consisted of either two or three, 0.375 or 0.5 in. diameter sheathed cables. Each cable consisted of seven, 7-wire steel strands with a guaranteed ultimate strength of 250,000 psi. All anchors were either proof or performance tested to 1.25 times the design load and locked-off at about 70 percent of the design load in accordance with the recommendations for prestressed rock and soil anchors by the Post-Tensioning Institute, (1980).

PHASE 2: ALLOWABLE VIBRATION CRITERIA AND VIBRATION MONITORING PROGRAM

The objectives of Phase 2 were to assess the risks that vibrations from the demolition could have on the adjacent facilities, establish allowable vibration criteria, and develop a vibration monitoring program to be implemented during demolition. The risks were evaluated by gathering information from the literature, reviewing our files, discussing the project with specialists in the design, construction, and beer industry and reviewing data from measurements taken at other demolition sites such as the Sheraton Hotel once located at 9th Street and Cole Avenue in downtown St. Louis, Missouri. Also, information obtained during the assessment of the response of the Los Angeles brewery to the 1972 San Fernando earthquake was used. A summary of typical vibration information is included in Plate 1. A peak particle velocity of 1.0 inches per second was recommended as the maximum allowable vibration measured on the stockhouse floors.

PHASE 3: DEMOLITION AND VIBRATION MONITORING PROGRAMS

Demolition Approach

The demolition and vibration monitoring program was developed based upon the results of the Phase 1 and 2 studies. The demolition of the Borsari Cellars was done during the period from July 14, 1993 to March 21, 1994. Prior to demolition, the Borsari Cellars were isolated from Stockhouses 12 and 13 by manually jack-hammering and sawing reinforced concrete structural ties and slabs and walls. This was done to minimize the risk of the transmission of vibration through the structural members. The contractor used a crane-mounted 5-ton headache ball with a drop or swing distance of about 50 ft. The demolition started on the south wall of Stockhouse 10 and 15, and proceeded to the north in a wave-like fashion. The lower levels of the cellars were filled with crushed demolition debris up to an elevation approximately level with the surrounding grades of Bud Alley and 11th Street. Due to the presence of important utilities in Bud Alley, and the plant requirement of maintaining traffic in the alley, the vertical 2-ft thick basement walls of the cellars

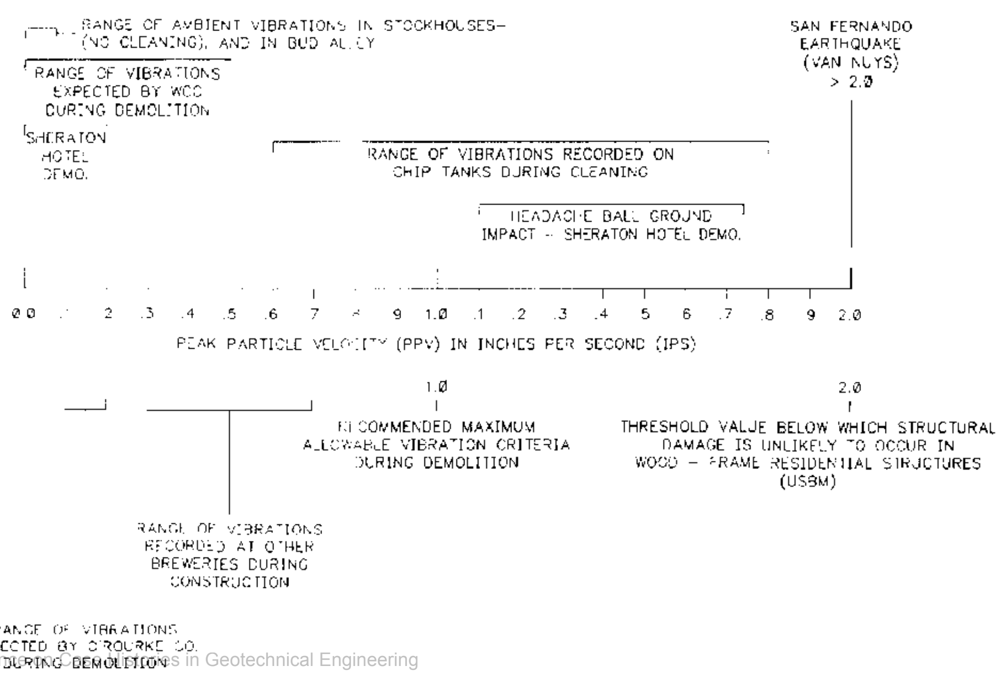

Plate 1: Spectrum of Demolition Vibrations

were left in place along the existing tunnels in Bud Alley. The demolition debris in the basements provided a temporary restraint along Bud Alley to prevent sloughing. Basement walls and debris were eventually removed and replaced by a temporary soldier beam and lagging shoring system at selected locations. Compacted structural backfill was placed in the basement area up to an elevation approximately equal to the finished floor elevation of the planned new stockhouse.

Demolition Monitoring Plan

A four phase demolition vibration monitoring plan based upon the demolition contractor's approach was adopted. A Geosonics SSU 1000D and an Intel Blastmate seismographs were used. The monitoring plan was intended to be as explicit as possible and at the same time provide some flexibility, particularly if the 1.0 ips PPV threshold value was exceeded or if the demolition contractor altered their plan. When possible, one seismograph was placed in a basement or lower floor of a stockhouse that was to remain, and one seismograph was placed on an upper floor approximately above the first seismograph. The intent was to evaluate vibration characteristics at different floor levels.

Since the demolition occurred in a northerly direction, the initial primary demolition seismograph locations were as shown in Figures 5, 6, 7, and 8 for Phases I, II, III, and IV of the demolition, respectively.

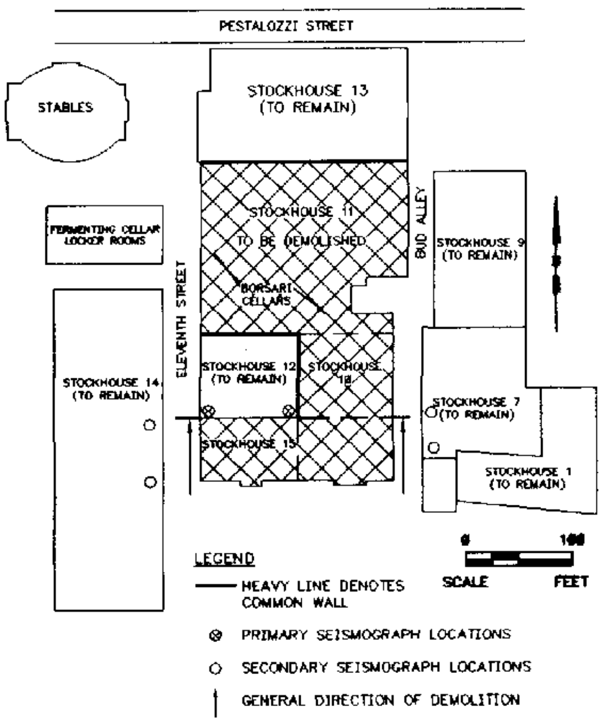

Figure 5: Seismograph Locations, Phase I.

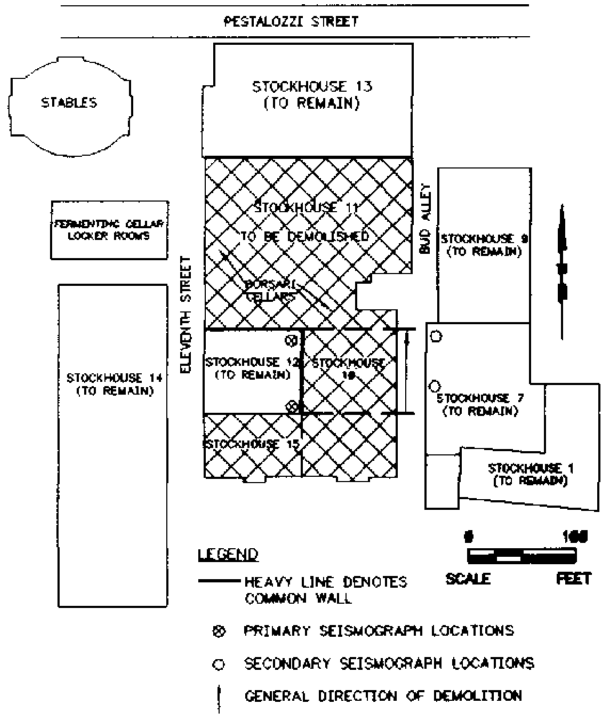

Figure 6: Seismograph Locations, Phase II.

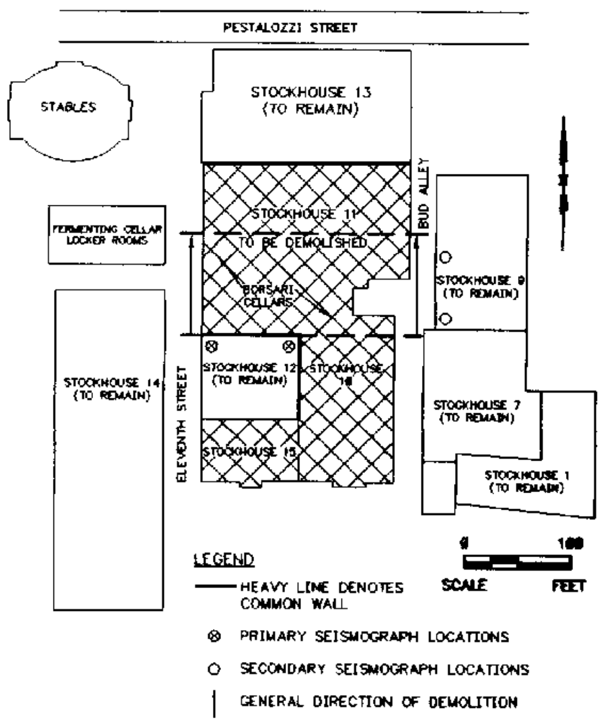

Figure 7: Seismograph Locations, Phase III.

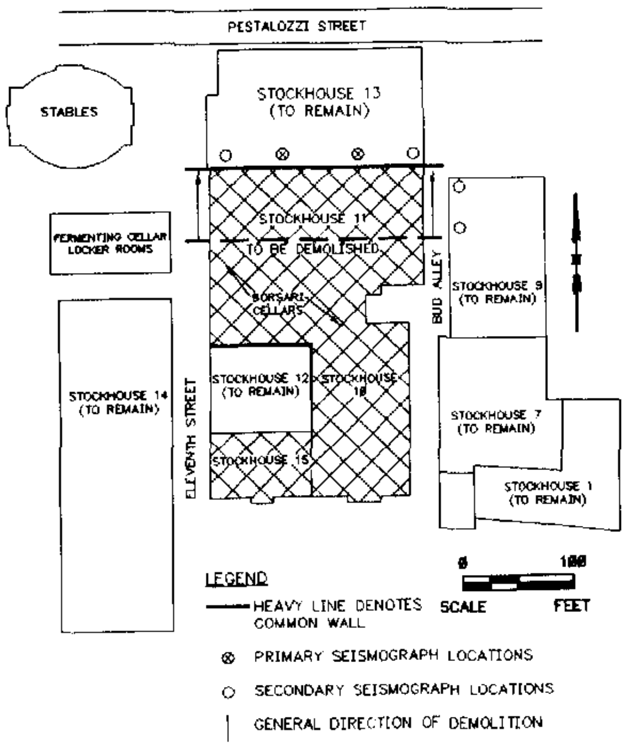

Figure 8: Seismograph Locations, Phase IV

The seismographs remained in the indicated primary locations until the demolition proceeded to a predetermined stopping point for each phase. A seismograph was periodically placed near the respective secondary monitoring locations. Secondary seismograph readings were taken on the upper floors of Stockhouse 12 to evaluate any amplification occurring within the structural frame and tankage in the stockhouse.

DISCUSSION

Readings at the upper floors were generally similar or lower than readings experienced at the lower floors. Demolition vibration data recorded for the entire demolition program are summarized in Table 2. These data were compared with those obtained during Phase I and were found not to be significantly different in magnitudes.

Three isolated readings with RPPV values greater than 1.0 inches per second were recorded during demolition. Immediate corrective measures such as reducing the swing distance of the headache ball were instituted. On each occasion, demolition then continued below the threshold level. It is noted that these vibrations are transient dynamic loads and therefore a RPPV value above a chosen threshold will not have the effect of a steady-state vibration of the same magnitude. It is also noted that a vibration reading with a RPPV close to 2 inches per second was recorded during the cleaning of the glass-lined tanks and no damage to the tanks was observed.

During the demolition qualitative visual assessments were performed to monitor: 1) the condition of the roof and sides of the north tunnel located just to the south of Stockhouse 10; 2) the condition of the roof and sides of tunnels located under the alley immediately adjacent to the east wall of Stockhouse 10; and 3) the condition of the roof and sides of two tunnel systems located under the alley between Stockhouses 9 and 11 (see Figure 1). No apparent damage was observed during these assessments. After completion of the demolition, the project site configuration was as shown in Figure 9.

Table 2: Ranges of Recorded Vibrations in Stockhouses During Demolition

Demolition Phase:	Phase 1			Phase II		Phase III	Phase IV		
Date:	7/14/93 - 11/16/93			8/10/93 -8/23/93		8/24 -11/2/ 1994	(9/14/93 - 1/11/94)		
Stockhouse No.:	12	14	7	12	7	12	13	13	9
Monitoring Location:	Primary	Secondary	Secondary	Primary	Secondary	Primary	Primary	Secondary	Secondary
Vibration Parameter									
PPV, (mips)			20 - 57			156 - 718	50 - 260	20 - 840	20 - 142
PD, (mils)	0.05 - 422		0-05 - 5.38			0.4 - 13.8	0.12 - 1.03	0.05 - 221	0.05 - 0.6
PPA (%g)							0.03 - 4.4	0.02 - .53	0.03 - 0.17
Freq (Hz)							17 - 86	0.4 - 86	16 - 100
RPPV, (mips)	20 - 700	20-60	200 - 640			653 - 766	110 - 260	100 - 840	74 - 163
Monitoring Duration (days)	25	1	11	9	0	19	16	19	6
No of Recordings	56	11	17	0	0	5	27	105	16
PPV Trigger level (mips)	20-500	20	150-200	500	500	100 - 500	20 -500	100 - 200	70 -200

Note : 1.0 mip = 0.001 inch per second ; 1 mil = 0.001 inch

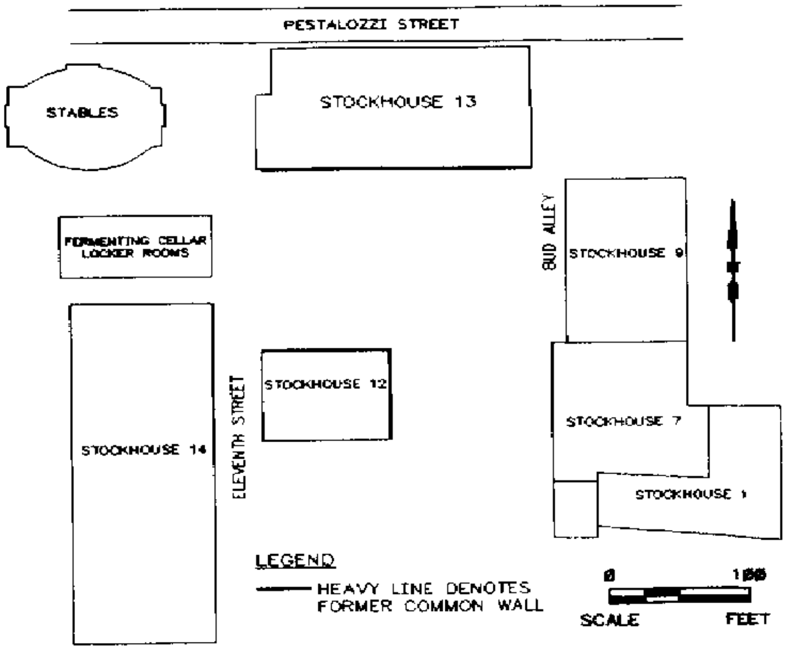

Figure 9: Site Plan After Demolition and Backfilling

CONCLUDING REMARKS

An allowable demolition vibration criteria based on a RPPV of 1.0 inches per second for the threshold value was used for this project. A heavily reinforced concrete structure was demolished with no damage to adjacent stockhouses, glass-lined beer tanks in the stockhouses, and to several underground tunnels on the project. Based on the data obtained in this study, it was observed that although daily routine activities continued during demolition, the vibrations measured before and during demolition were not significantly different. It is believed that for these types of structures, demolition can be done without significant damage to adjacent buildings when the resulting PPV vibrations are limited to less than 1 inch per second. This threshold will likely be different for different types of structures.

ACKNOWLEDGMENTS

We wish to thank Anheuser-Busch Companies, Inc. Corporate Engineering personnel for their assistance and cooperation during the project and the preparation of this paper. Thanks are also due to our Woodward Clyde colleagues who helped in one way or another.

REFERENCES

Post-Tensioning Institute, "Recommendations for Prestressed Rock and Soil Anchors", 1980.