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Dynamic Testing of Materials

L. J. MITCHELL, *U. S. Bureau of Reclamation*

THIS paper describes dynamic tests of both sonic velocity and resonant frequency types. Sonic velocity tests were made to evaluate the effect of both air-filled and water-filled cracks on transmittal of signals through concrete. Low-frequency signals, especially those groups caused by hammer blows, showed little effect from either narrow or water-filled cracks. As would be expected, any attenuation or delay was increased as the crack size increased. An increase in the frequencies composing the signal also increased the effect of both air- and water-filled cracks.

Another series of investigations consisted of sonic-velocity measurements on masses of sand, gravel, combinations of sand or gravel with water, and freshly mixed concrete. Decreasing particle size reduced the measured sonic velocities. The presence of water in the above mixtures increased the sonic velocity in all cases. Measured velocities in this series were much below velocity of sound in air and, when compared to data presented by other investigators, led to the conclusion that the apparent velocity through a loose mass is dependent upon the character and frequency of the signal. These results led to the investigation of various pickups and circuit components which demonstrated that ordinary equipment components always produced time delay factors in the measurement. Some of these variables are caused by surface conditions, power and type of the original signals and the mounting of pickup components.

Resonant-frequency dynamic tests were compared to sonic velocities. Flexural and torsional resonant frequency testing on rock cores revealed unusual reactions. Some indications of heterogeneity and loose structure were evident even in apparently sound specimens. Irrational values of Poisson's ratio, which were frequently indicated by these cores, are discussed.

● MANY investigators have been interested in dynamic, or non-destructive, testing of materials during the past several years. We are concerned with two general methods of test which have been developed, along with special equipment for each. The first method, testing by sonic vibration or resonant frequency, has been quite useful but is applicable

for laboratory specimens only. The second method, testing by wave velocity, is divided into two types, pulse-velocity technique, and interval-timer technique. The pulse-velocity equipment provides visual measurement, using pulses of ultrasonic vibration. The interval-timer equipment is essentially a condenser chronograph which uses sonic-frequency groups of vibration usually produced by a hammer. Either of these velocity measurements can be applied in the field and in the laboratory. The more-recent investigations seem to favor the pulse-velocity method, because the visual indication allows the operator to observe the actual beginning of the transmitted wave. Many excellent descriptions of both equipment and tests have been published during the past decade.

This paper presents data from four studies in nondestructive testing. These studies are: (1) the effect of cracks upon dynamic testing, (2) time delays in pickup units, (3) wave-velocity tests through plastic or loose masses, and (4) dynamic tests of rock cores. A few of the pitfalls in this work are discussed and some unique data are presented.

EFFECT OF CRACKS UPON DYNAMIC TESTING

General Considerations

The effect of cracks in concrete structure upon dynamic testing, in which velocity is measured, can be of two general types, either an attenuation of the signal or a delay in the transmission resulting in a longer measured time and, hence, lower indicated velocities. Reductions in wave velocity caused by cracks can come about in two ways, only one of which is likely to be serious in measurements with either soniscope or microtimer equipment. The greatest reduction in indicated velocity due to cracks is caused by the signal being transmitted around the cracks. A minor reduction in measured velocity can be caused by crack filling material. There is also the possibility of an indicated longer time or lower velocity due to attenuation of the signal or changing of the wave front shape in crossing cracks. This could cause a delay in detection of the signal.

Test Procedure

One investigation of the effect of cracks was accomplished by using four 6- by 12-inch cylindrical concrete specimens placed end to end with separate pieces of cork beneath each and measured gaps between adjacent specimens to simulate cracks perpendicular to the common axis. A strip of soft rubber about 1/32 inch thick and 1½ inches wide was then wrapped and clamped watertight around adjacent ends of the cylinders, and the gaps filled with water to simulate waterfilled cracks. The electronic interval timer (*1*) (microtimer) was used to measure the time travel of longitudinal vibrations created by the impact from a pendulum-type hammer upon the end of the first specimen. This hammer consisted of a 282-gram iron head upon a 12½-inch lever arm.

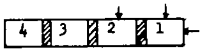
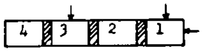
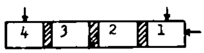
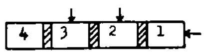
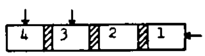
The effect of extreme fracturing and fine cracks was investigated by using a 6- by 12-inch concrete cylinder which had been loaded to failure in the triaxial testing machine. This specimen was placed end to end between two normal cylinders of concrete and completely enclosed in a rubber sleeve. The sleeve was then clamped water-tight to the normal cylinders. Tests were made in the dry condition and periodically over a 2-week period after the sleeve surrounding the fractured specimen was filled with water.

Since the accuracy of velocity measurements is somewhat dependent upon the amplitude of vibrations being measured, oscillographic records of the wave fronts were taken as the wave passed through the specimens. The pickups were phonograph-type, piezoelectric crystals normally used with the interval timer.

Discussion of Results

The velocity measurements taken over an 8-inch length on the individual cylinders ranged from 11,900 feet per second to 12,500 feet per second, with an average value of 12,300. Velocity measurements upon combinations of specimens separated by gaps of various thickness gave the results shown in Table 1. The vertical arrows in the diagram, lefthand column of Table 1, indicate the location of the vibration pickups between which the velocity was measured. The horizontal arrow indicates the point of impact of the pendulum hammer which initiated the vibrations. Water-filled gaps between the specimens are represented by cross-hatched areas. Columns marked "uncompensated" are average velocities over the total distance between the two pickups. The columns marked "compensated" have been corrected to give the average velocity through the concrete, exclusive of the separating gaps, assuming the velocity of sound in water to be

TABLE 1
VELOCITY MEASUREMENTS ACROSS WATER-FILLED GAPS

Location of pickups	Width of gap between specimens							
	1/16″		1/8″		1/4″		1/2″	
	V Uncomp	V Comp	V Uncomp	V Comp	V Uncomp	V Comp	V Uncomp	V Comp
	12,200	12,300	12,000	12,100	11,800	12,200	10,700	11,300
	12,300	12,400	12,300	12,500	11,900	12,200	10,900	11,400
	12,100	12,200	12,300	12,400	11,700	12,000	10,300	10,800
	11,900	12,000	12,300	12,400	11,800	12,200	10,600	11,000
	12,100	12,200	11,700	11,700	11,000	11,300	9,000	9,300

4,900 fps. The values shown in Table 1 are the average of two values taken with the cylinders reversed to minimize the effect of variation between individual cylinders.

Figure 2 and Table 1 show that longitudinal vibrations are transmitted across water-filled cracks, with little change in velocity or amplitude up to a crack width of about 1 percent of the path length traversed. Cracks of 1/4-inch width, about 2 percent of the path length, produced definite, though not large, reductions in velocity. Cracks of 1/2-inch width produced serious reductions in measured velocity. The indicated velocity decreased, both with the length of the measured course and distance from the initiated impulse. This was partly due to measured velocity being slightly dependent on the signal amplitude.

Figures 1 to 4 are oscillographic records of the vibration wave front as detected by the piezoelectric crystal under various conditions of test. The test conditions are indicated by diagrams similar to those in the left-hand column of Table 1. These oscillographic records show that high-frequency components of the initial steep wave front disappear almost completely when measured across two 1/16-inch water-filled gaps. The lower frequency, regularly shaped wave remained strongly present. West (*2*) shows that the attenuation factor is a function of the frequency of vibration. However, he believed that the variation is not great enough to account for the observed loss of the higher frequencies in the signal group. The oscillograph limited the record to those frequencies below 9,000 cycles per second. Figure 5 supports this belief by showing that the high-

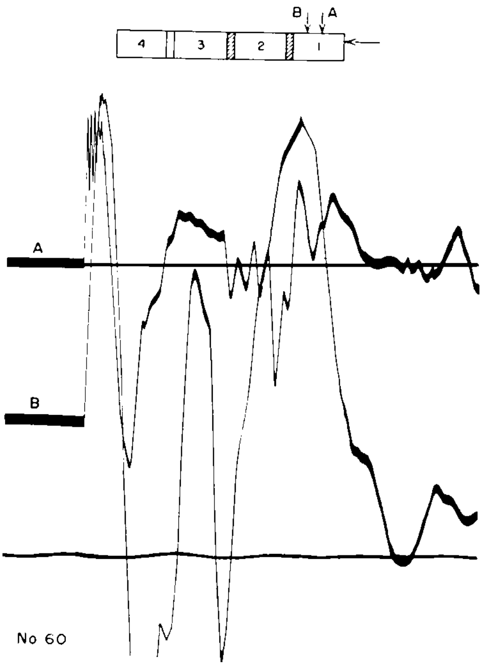


Figure 1. Oscillogram of vibration wave front.

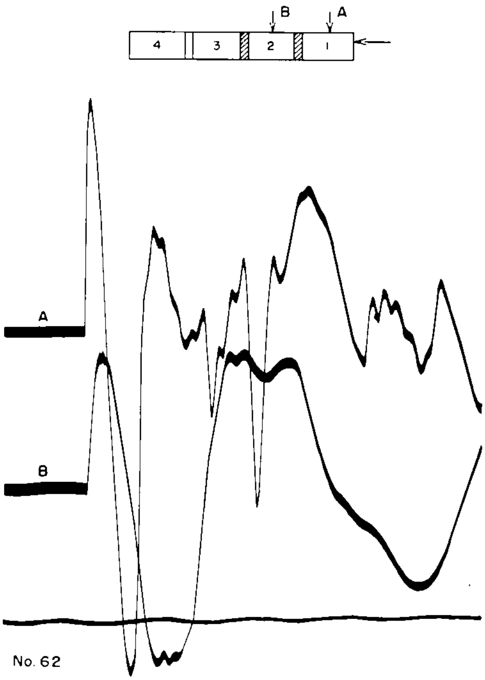


Figure 2.

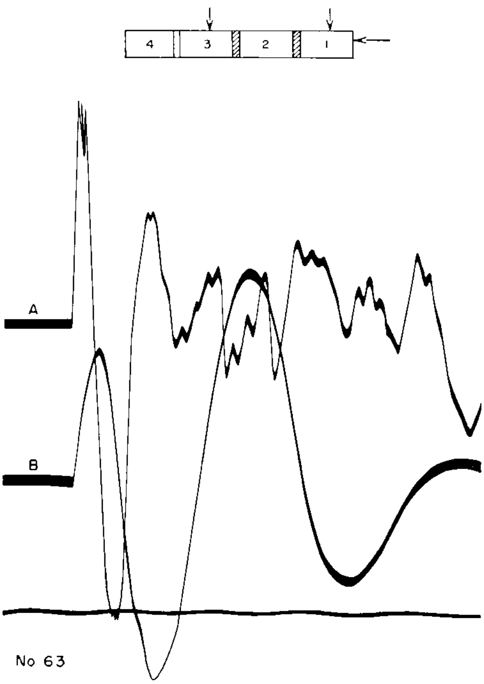


Figure 3.

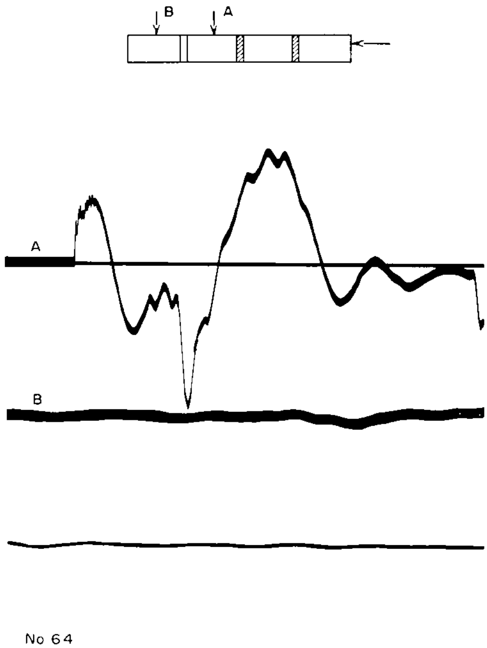


Figure 4.

frequency components are not greatly reduced over a 2-foot distance in a sound 4- by 4- by 40-inch concrete bar without cracks. Figure 4 shows that an air gap ($\frac{1}{16}$ inch) reduces even the lowest frequency to an unusable small amplitude. Air gaps to pass vibrations suitable for accurate measurements must be extremely small.

It was possible to test across air-filled cracks of the magnitude involved in the crushed specimen. However, the saturated condition was of primary interest. Wave-velocity tests on the crushed specimens are recorded in Table 2. An increase of approximately 50 percent in velocity was observed during the 2-week period the specimen was immersed in water. This increase is principally due to absorption of water into the air voids.

While low-frequency signals will usually cross cracks instead of detouring them, ultrasonic signals have short range in material that is badly deteriorated and are not likely to cross a significant crack.

TIME DELAYS IN PICK-UP UNITS

Preliminary tests indicated that the phonograph-type crystal pickups had low time delay and were relatively uniform. The calibration of a standard bar for this investigation was

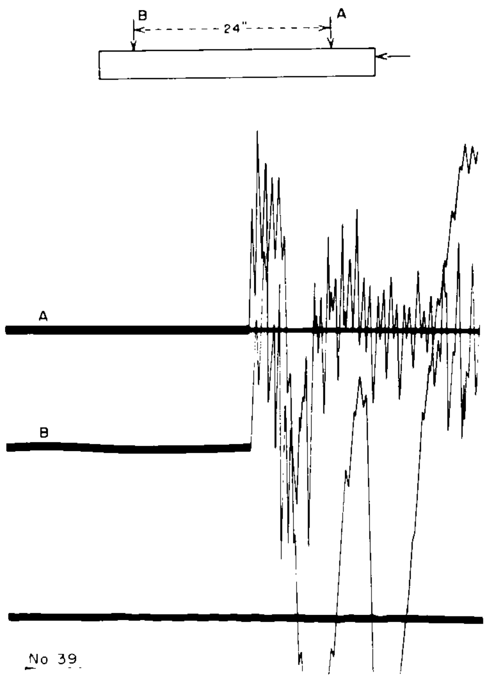


Figure 5.

TABLE 2
VELOCITY MEASUREMENTS ON CRUSHED CYLINDER

Location of pickups	Date	Velocity (ft per sec)
	9-17-47	6,100
"	9-18-47	6,500
"	9-19-47	6,400
"	9-22-47	7,600
"	9-23-47	7,800
"	9-24-47	8,300
"	9-25-47	8,800
"	9-30-47	9,400

TABLE 3
COMPARISON OF PICK-UP UNITS
Time delays in microseconds

Hammer stroke	5°	10°	30°	60°
Pick-up				
Phonograph (crystal)	62	61	52	46
Carbon microphone	63	57	42	40
Electric transducer	87	77	70	62
Vibromike VM-1 (rubber covered)	111	91	67	58
Vibromike VM-1 (without cover)	95	70	6	47
Phonograph (crystal) (with amplifier)	27	26	19	17

Note: The hammer used in all tests was a 282-gram pendulum with an arm 12½ inches long.

made with phonograph-type crystal pickups. This calibration consisted of many microtimer measurements on a heavy brass bar with careful reversing of pickups. The correct velocity for the brass was determined to be 11,500 fps. The selection of pickups for use with the microtimer and later, the timerscope (see Appendix A) was made after careful investigation of six types of pickups.

Investigations of pickup delays were made by replacing the first pickup at the end of the bar where the hammer struck with an electric contact and by locating the second or test pickup at a distance of 46 inches from the initiated impulse. Various controlled impulses were delivered by dropping the 282-gram pendulum hammer through various arcs on a 12½-inch arm. The data in Table 3 show that the time delay changed nearly 100 percent when the hammer fall was varied from 60 deg. to 5 deg. The data plotted in Figure 6 show that the time delays approach constant values for the higher arcs of hammer fall. The covering on the Vibromike pickup caused an appreciable difference in delay. The phonograph pickups and the electronic transducer were affected by type and length of probe. Recorded delays are for a minimum length of metal probe. Some supposedly identical pickup units had large differences in delay.

WAVE VELOCITY TESTS THROUGH PLASTIC OR LOOSE MASSES

Tests on Fresh Concrete

Whitehurst (*3*) and Cheesman (*4*) reported in 1951 some indication of a relationship between wave velocity by soniscope measure-

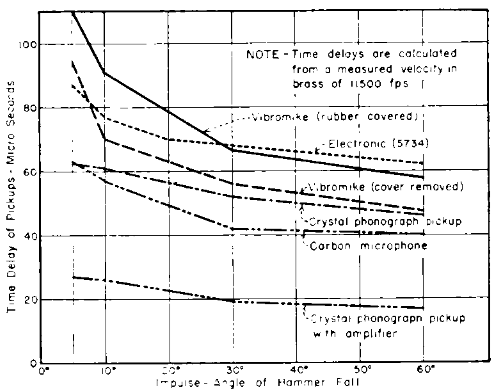


Figure 6. Comparison of possible micrometer pickups.

ments and setting time of concrete. Bureau of Reclamation microtimer and later timerscope tests were made to determine the feasibility of using wave velocity measurements as an indication of the time of set of concrete. It was hoped that these tests would help evaluate the effect of accelerators and retarders on the hardening of concrete and determine the earliest time at which forms could be safely removed.

A cubic form was built especially for this work, 1 cubic foot in size, with windows in two opposite sides closed by rubber diaphrams. Small metal plates, containing metal studs, were attached through the centers of the rubber diaphrams for initiating and picking up signals. The 282-gram pendulum hammer swinging on a 12½-inch arm provided a source of controlled uniform signals.

The velocity tests through fresh concrete, using the microtimer, were started 10 to 20 minutes after mixing. Although some difficulty was encountered in getting good signals through the concrete at these early ages, satisfactory signals were usually obtained 30 minutes to 1 hour after placing. Initial velocities of 400 to 600 fps. were indicated. These velocities increased very slowly up to 2 or 2½ hours, after which a relatively rapid increase took place.

As it was believed that the true minimum velocity could not be less than that of water (determined at 4,750 fps. with the microtimer), special investigations were made to assure that Rayleigh waves or signals transmitted through the form were not being measured instead of signals through the test mass.

In order to facilitate further study of this problem, the Reclamation laboratories developed a new apparatus, timerscope (Appendix A), which allows visual observation of the waves and can be read to very close tolerance on an expanded time scale. This device, while still using hammer blows, either manual or repeated by a mechanical device, provides visual detection and matching of the signal with a timer pip on the oscilloscope. The oscilloscope screen is blank, except when a signal is being transmitted. The scope-sweep and timer action are triggered by electrical contact the instant the pendulum hammer strikes the specimen. Appendix A gives a more detailed description of the operation of the timerscope along with block diagram, Figure A.

Tests with the timerscope verified the low velocities through fresh concrete.

Tests by other investigators using hammer-blow signals agree closely with the above findings. Andersen and Nerenst (*5*) report velocities of about 600 fps. in fresh concrete. Their signals were initiated by a small electric hammer operated from a 6-volt battery. The measuring equipment was a condenser chronograph (essentially the same as a microtimer) and the pickups were calibrated for delays and differences. They found that the test results depend not only upon the steepness of the wave front and hence the energy of impact, but also that more energy is needed for larger test masses. Their small electric operated spring hammer was not only inadequate for testing a hardened 7-inch pavement slab, but over a path length of more than 100 cm. measurements suddenly corresponded to the velocity of slower waves, presumably Rayleigh waves. A 300-gram hammer, working as a pendulum, gave more reasonable and uniform results.

L'Hermite (*6*), whose data agree with the above, used an inner and outer pair of independent box forms and a small cam and spring operated hammer with condenser chronograph-type instruments. He concluded that the measured wave was an energy wave similar to one transmitted through a series of coupled pendulums.

However, investigators (*3*, *7*, *8*) using ultrasonic pulse signals and soniscope instruments have found initial velocities of 2,000 to 3,000 fps. at 2 to 3 hours after mixing. Leslie and Sturrup (*8*) state that even when soniscope transducers are cast into the concrete and readings taken almost immediately after mixing, the indicated velocity remained almost constant at about 2,000 fps. during the first 3 hours.

Discussion of Data

Our tests yielded no sharp break in the curves to indicate the point of initial or final set, but changes in slope did occur. Typical curves are shown in Figure 7. All data indicated that the wave velocity through fresh concrete was low at early ages, increased rapidly after about 2 to 3 hours and continued until between 6 and 12 hours. Variation in time depended upon the cement mix and quantity

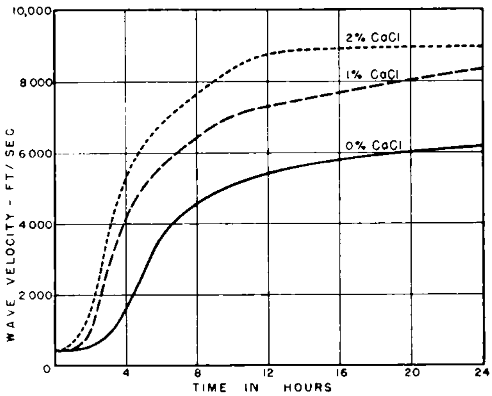


Figure 7. Time-of-set investigations.

of accelerator. The addition of calcium chloride to the concrete definitely hastened the appearance of the slope changes. The general changes in slope of the curves roughly checked the times of set as indicated by standard neat cement tests. It is only possible to state that the correlation is of the same general order.

Jones (7) stated that wave-velocity tests on fresh concrete may be useful as a method for estimating strength development to determine when forms may be safely removed. He found a usable relationship between velocity and strength at early ages, provided great accuracy is not needed. Use of this relationship requires a new calibration for each aggregate and each mix design.

The definite grouping of the above test results according to the type of equipment proves that the signal characteristics influence the indicated wave velocity in fresh concrete.

Velocity Tests Through Masses of Loose Aggregate.

Attempts to explain the extremely low-velocity measurements through fresh concrete led to a series of tests in which aggregates from No. 50 sand to 6-inch cobbles were placed in the forms and tested in both wet and dry conditions. The same form box and technique, both microtimer and timerscope, were used to measure the velocity of signals in the aggregate masses as were used in the previous concrete tests. All of the aggregate masses were tested immersed and then in a drained state. The larger size fractions were tested both with random filling and with careful placing of rocks so as to have solid direct contact between adjacent pieces. All tests were made with a 60 deg. fall of the pendulum hammer.

Discussion of the Velocity Data on Aggregate Masses

The results of this test are shown in Table 4 and Figure 8. Extremely low velocities, similar to those for fresh concrete, were obtained on fine sand. The solid line in Figure 8 represents average values on the various sizes of aggregates plotted to the average particle diameter.

It was concluded that these extremely low velocities in the fine aggregates could not possibly represent sonic or elastic wave velocities. An energy wave would travel through the mass in a manner similar to that set forth in the approximate method of computation shown in Appendix B. This solution offers an explanation of the low velocities observed in loose masses, and agrees, in general, with L'Hermite's (6) conclusion that the signal is that of an energy wave transmitted in a manner similar to the action of coupled pendulums. The dotted line in Figure 8 was plotted from velocities computed according to this assumption and approximate solution. Complete agreement should not be expected, as the solid line for the aggregate is drawn through points representing the mean size of the various screened aggregates, which are by no means spheres and are not ideally located within the mass. Also, as aggregate size decreased, test difficulty increased, and a wider spread in results occurred. The solid curve apparently

TABLE 4
MEASURED VELOCITIES IN SAND AND GRAVEL
12-inch Path Length

Aggregate size	Diameter inches	Velocity	
		Flooded	Drained
No. 50	0.018	540	530
No. 16	.070	610	640
No. 4	.28	1,020	920
¾ to 1½ inch	1.125	2,020	1,960
3 to 6 inch	4.5	5,250	5,250
3 to 6 inch	4.5	9,800*	
3 to 6 inch	4.5	9,540*	
3 to 6 inch	4.5	11,900†	

* Rocks were rearranged to make the path of the impulse between 80 and 90 percent solid rock.

† The direct path of the impulse was solid rock wedged between the windows.

Note: Five additional large cobbles from the same source gave an average velocity of 12,800 feet per second.

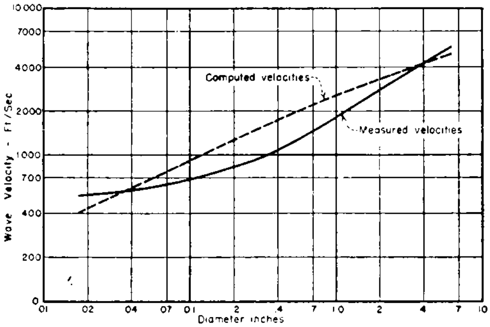


Figure 8. Indicated velocities through wet sand and gravel.

approaches a minimum value for small-sized aggregate, but it seems more likely that the limit is a function of the equipment. Group frequencies of large amplitude gave one minimum value for velocity in fresh concrete, while all ultrasonic pulses gave another but higher minimum velocity. The indicated minimum velocity depends upon the signal frequency or amplitude and possibly upon the test mass.

DYNAMIC TESTS OF ROCK CORES

Tests and Procedures

During the past 3 years, many NX and BX size (approximate 2⅛ and 1⅝ inches in diameter, respectively) rock cores have been tested for dynamic properties by means of the resonant-sonic-frequency method ASTM C-215. This program provides designers with more-complete information on the characteristics of foundation rocks from proposed projects. These cores include a great variety of rock types from many parts of the western United States.

The usual and desired length of specimens for rock-core dynamic tests was established at four times the diameter. This uniform length of specimen simplified test procedure and computations in addition to providing two normal static or triaxial test specimens.

In testing, specimens were supported on a pad of sponge rubber and contacted at one end by the metal prod of the driving unit. A phonograph-type crystal pickup was used to detect the vibrations which were examined for resonant amplitude and node points by means of a cathode-ray oscilloscope. The oscilloscope was used as an indicator, because it was frequently desirable to observe the wave form of a vibrating specimen and sometimes advisable to check the frequency ratio between driver and pickup units. This type of examination helped greatly to detect spurious vibrations in some specimens, which provided an apparent multitude of resonant frequencies.

Specimens of these proportions will usually vibrate in either a flexural mode or a torsional mode whenever subjected to excitation at the proper frequency, regardless of the direction or location in which the excitation is applied.

Location of resonant frequency is always accomplished with the pickup at the end of the specimen opposite the driving unit. The oscillator which feeds the driver unit is tuned until a peak amplitude indicating a resonant frequency is observed. Then the pickup unit is moved from its initial position at the end of the specimen to a point approximating the flexural node location. If any indication of a possible node proximity is observed, its actual location and characteristics are checked. The pickup is then moved to contact the specimen near the center for checking a possible torsional node or some indication of a higher mode of vibration. If broad, incomplete, or displaced nodes are observed, it is advisable to check the general shape of the amplitude curve along the specimen. Marked irregularities or flatness in this curve frequently indicate combination or spurious frequencies. When in doubt, the wave form should always be checked, as a distorted wave usually accompanies spurious or combination frequencies. When resonant peaks are apparently of extreme width or doubled, a careful recheck is in order, as this characteristic frequently indicates the presence of a combined frequency. Results that are in doubt can frequently be checked by rotating the specimen 45 deg. to 90 deg. or even 180 deg. about its axis and retesting. Torsional-frequency readings should check and flexural frequency readings should check within 2 percent on cylindrical specimens which appear sound and homogeneous.

The cores in this program were usually tested in air-dry, oven-dry, and vacuum saturated conditions, after which they were redried to approximately 75 percent saturation for static test and triaxial shear test. Approximately a third of the specimens were tested by

TABLE 5

Core No.	Sonic resonance dynamic tests: Oven dried E	Oven dried G	Oven dried μ	Saturated E	Saturated G	Saturated μ	Saturated % H_2O	Static load tests: E	μ	% H_2O	Remarks
					Hypersthene Andesite						
1	8.0	3.3	0.20	6.5	2.7	0.21	2.40	9.1	0.11	1.80	
9	7.7	3.2	.19	6.6	2.8	.20	2.05	8.9	.25	1.54	
23	7.5	3.2	.19	6.7	2.7	.23	1.69	7.1	.17	1.27	
29	8.0	3.4	.18	7.1	3.0	.19	1.10	7.8	.17	0.82	
35	7.7	3.2	.18	7.0	2.8	.24	1.60	6.8	.16	1.20	
41	7.9	3.4	.17	7.0	2.9	.20	1.02	7.9	.20	.76	
					Basalt						
68	8.9	3.7	.20	9.1	3.7	.22	2.42	10.0	.24	1.82	Vesicular with average vesicule 0.5 mm
69	8.8	3.6	.20	8.8	3.6	.21	2.66	8.6	.21	2.00	
70*	8.8	3.6	.20	8.8	3.6	.21	2.66	8.9	.23	2.00	
71	8.9	3.7	.20	8.9	3.7	.21	2.50	8.9	.18	1.88	
72*	8.9	3.7	.20	8.9	3.7	.21	2.50			1.88	
75	2.8	2.3	−.40	3.4	2.5	−.32	3.77	4.6	.06	2.83	
54	3.7	1.6	.16	5.1	†	†	4.83	2.5	.04	3.62	1.0 mm vesicules with thin filled fractures
59	6.1	2.7	.14	7.0	2.9	.21	5.19	5.0	.15	3.89	
12	8.3	3.4	.21	9.0	3.5	.30	4.34	7.4	.18	3.26	Vesicular and slightly porphyritic
15	7.7	3.2	.20	8.0	3.3	.22	4.84	7.5	.22	3.63	
18	7.7	3.2	.19	8.1	3.3	.22	4.28	7.6	.23	3.21	
22	8.6	3.6	.21	8.8	3.6	.21	4.37	11.6	.09	3.28	
27	8.0	3.4	.18	8.3	3.5	.19	3.14	7.7	.23	2.36	
6	6.4	2.7	.19	6.8	2.8	.24	6.19	7.7	.20	4.64	Like Swiss cheese
52	5.6	1.8	.53	5.5	2.3	.19	1.08	5.0	.36	0.81	Many cemented cracks
53	4.5	1.3	.68	6.1	2.4	.29	1.58	6.2	.12	1.18	
59	1.5	0.8	−.07	3.2	1.6	.05	1.75	3.0	.12	1.31	
67	3.3	1.6	.01	5.2	2.4	.09	1.73	7.1	−.26	1.30	
68	2.4	1.1	.05	3.8	3.0	−.36	2.97	2.4	.02	2.23	Many cemented cracks
69*	2.4	1.1	.05	3.8	3.0	−.36	2.97	2.4	.09	2.23	
71	0.7	0.4	−.18	0.8	0.5	−.18	12.01	0.4	.02	9.01	Badly deteriorated
72*	0.7	0.4	−.18	0.8	0.5	−.18	12.01	0.2	0	9.01	
					Claystone						
21	1.5	0.7	.10	‡	‡	‡	‡	1.1	.02	‡	Silty
76	1.7	0.7	.21	‡	‡	‡	‡	1.6	.09	‡	
48	0.9	0.6	−.27	‡	‡	‡	‡	0.7	0	‡	
16	2.3	0.9	.23	‡	‡	‡	‡	0.6	.04	‡	Mixed
17*	2.3	0.9	.23	‡	‡	‡	‡	2.2	.04	‡	
64	2.9	1.3	.13	‡	‡	‡	‡	2.6	.05	‡	
					Diorite Gneiss						
16	12.0	5.2	.15	12.0	5.2	.15	.09	7.9	.13	.07	
23	13.7	5.8	.19	14.0	5.9	.19	.07	10.0	0	.05	
					Granite						
38	6.1	2.6	.18	6.4	2.8	.15	.44	5.3	.17	.33	
39*	6.1	2.6	.18	6.4	2.8	.15	.44	4.4	.12	.33	
47	4.6	2.1	.11	4.7	1.9	.20	.44	3.2	.10	.33	Altered
					Greywacke						
2	4.3	2.1	0.03	3.6	1.8	0.07	4.20	1.6	0.02	3.15	Coarse grained
3*	4.3	2.1	.03	3.6	1.8	.07	4.20	1.6	0	3.15	
6	4.5	2.2	.04	3.3	1.8	−.07	3.99	2.0	.03	2.99	
7*	4.5	2.2	.04	3.3	1.8	−.07	3.99	1.9	.03	2.99	
13	3.6	1.9	−.04	3.0	1.3	.12	4.75	0.9	.02	3.56	
19	4.4	2.1	.04	3.6	1.8	.01	3.96	1.7	.03	2.97	
20	4.3	2.1	.01	3.8	1.9	0	4.31	1.3	.01	3.23	
30	4.9	2.3	.07	3.8	1.8	.08	3.79	2.1	.04	2.84	
101	4.3	2.0	.01	3.0	1.3	.15	4.09	1.4	.01	3.07	
117	4.5	2.1	.10	3.4	1.4	.18	4.29	1.7	.02	3.22	
57	4.9	2.1	.14	3.2	1.3	.21	4.62	1.7	.02	3.47	Fine grained
42	5.1	2.1	.22	3.2	1.3	.19	5.31	1.7	.03	3.98	
44	4.9	2.1	.18	3.5	1.3	.33	4.98	1.6	.05	3.74	
47	4.9	2.1	.16	3.2	1.3	.23	5.11	1.6	.05	3.83	
84	4.7	2.1	.13	2.9	1.3	.15	3.76	1.5	.01	2.82	
88	4.6	2.0	.13	3.5	1.3	.34	4.17	1.4	.04	3.13	
93	4.5	2.0	.13	3.5	1.2	.43	4.26	1.4	.02	3.20	

TABLE 5—*Continued*

Core No.	Sonic resonance dynamic tests							Static load tests			Remarks
	Oven dried			Saturated							
	E	G	μ	E	G	μ	% H_2O	E	μ	% H_2O	
Limestone											
4	11.6	4.4	0.30	11.2	4.2	0.35	0.10	11.1	0.31	0.08	Fine to medium grain 6-inch cores
6	12.0	4.5	.33	11.9	4.5	.33	0.22	12.2	.24	.16	
9	10.9	4.2	.29	11.0	4.1	.33	0.25	10.0	.16	.19	
11	11.2	4.3	.30	11.2	4.2	.34	0.38	12.5	.38	.28	Medium to coarse 6-inch cores
12*	11.2	4.3	.30	11.2	4.2	.34	0.38	10.2	.35	.26	
82	11.0	4.0	.25	8.9	3.4	.30	1.19	6.8	.15	.89	Stylolitic BX cores
87	8.6	3.5	.23	7.8	2.9	.37	2.75	5.2	.08	2.06	
92	7.5	3.1	.19	6.8	2.8	.20	3.71	4.8	.10	2.78	
5	11.5	4.4	.29	10.3	4.0	.29	1.92	10.0	.22	1.44	Fine grain BX cores
10	10.6	4.2	.25	9.6	3.7	.30	2.77	10.0	.30	2.08	
62	8.4	3.8	.10	8.8	3.7	.20	.35	9.1	.24	0.26	
40	10.1	4.2	.20	10.3	3.4	.29	1.28	7.2	.14	0.96	Chalcedonic
42	10.6	4.1	.30	9.2	3.7	.26	1.68	8.0	.18	1.26	This core BX
17	10.8	4.3	.26	11.0	4.3	.28	.41	10.0	.24	.31	Oolithic BX cores
29	6.2	2.9	.09	6.8	2.7	.25	.27	2.8	.08	.20	
69	7.1	3.1	.14	6.7	2.9	.14	.91	6.9	.21	.68	
23	5.7	2.4	.18	4.0	1.9	.06	6.23	3.6	.20	4.67	Porous BX core
26	6.3	2.6	.18	5.5	2.2	.26	5.01	3.3	.32	3.76	BX core
31	2.6	1.2	.06	2.0	0.7	.41	4.62	1.0	.05	3.46	NX core
75	7.8	3.1	.24	7.5	2.8	.34	2.05	5.0	.17	1.54	Medium grain BX core
Monzonite Porphyry											
6	8.6	3.6	0.20	7.9	2.9	0.36	1.66	5.6	0.18	1.24	
10	7.8	3.5	.11	7.8	3.2	.22	1.04	6.4	.18	0.78	
Phyllite											
4	2.7	0.7	.95	1.8	0.5	.73	9.95	4.2	0	7.46	
20	2.4	0.9	.32				4.51	1.3	0	3.38	
22	2.1	0.9	.22	1.0	0.4	.30	4.20	0.8	−.03	3.15	
23	3.9	1.0	.90	1.9	0.5	.79	5.92	1.8	−.07	4.44	
Sandstone											
42	4.4	2.3	−.02	3.7	1.7	.09	4.43	1.7	.10	3.32	
46	2.8	1.4	.03	2.1	1.0	.07	5.11	1.3	.05	3.83	
51	2.1	0.9	.12	1.5	0.4	.77	7.75	0.5	.08	5.81	
62	3.0	1.6	−.07	2.4	1.2	−.06	7.94	2.4	.05	5.96	
69	4.5	2.0	.12	3.3	1.6	.05	6.09	2.3	.04	4.57	
70	8.2	3.7	.10	7.2	3.2	.13	2.82	3.3	.07	2.12	
82	3.3	1.6	.06	1.9	1.1	−.15	6.36	1.8	.04	4.77	
83*	3.3	1.6	.06	1.9	1.1	−.15	6.36	1.6	.06	4.77	
Schist											
50	8.2	3.3	.23	8.4	3.2	.30	0.42	6.2	0	0.32	
75	7.1	2.0	.73	7.6	2.5	.50	0.45	4.0	−.60	0.34	
86	12.4	4.7	.32	12.1	4.6	.32	0.19	9.4	.15	.24	
Shale											
28	4.3	2.2	0	‡				3.6	0	‡	Laminated 6-inch cores
40	5.3	2.6	.03	‡				3.2	.11	‡	
30	4.2	2.2	−.04	‡				3.2	.16	‡	
11	2.1	1.5	−.29	‡				1.6	.03	‡	Quartzose NX cores
17	3.4	1.8	−.06	‡				4.4	.22	‡	
20	2.9	1.8	−.18	‡				1.6	.04	‡	
36	1.3	0.7	−.14	‡				0.5	—	‡	
2	3.6	2.0	−.08	‡				2.7	—	‡	Calcareous NX cores
13	1.8	1.5	−.40	‡				2.1	—	‡	
16	4.9	3.8	−.35	‡				1.2	—	‡	
22	2.8	1.8	−.21	‡				1.8	.02	‡	
24	3.8	2.2	−.14	‡				2.6	.03	‡	
33	4.2	2.3	−.09	‡				3.3	—	‡	
Siltstone											
40	5.4	2.2	.23	3.6	1.4	0.27	5.38	2.0	.08	4.04	BX cores
78	5.3	2.4	.10	4.5	1.7	.33	3.07	2.6	.02	2.30	
91	4.6	1.9	.20	2.7	1.2	.13	4.05	1.3	.07	3.04	
Tuff											
	0.3	0.2	−.25	‡				0.2	.06	‡	Lithic

* This specimen and the above were a single core during the dynamic tests.
† Dynamic tests were impossible in this condition.
‡ Tested in air dry condition only. The nature of this rock made oven dry tests or saturated tests impossible.

static methods for Young's modulus of elasticity and Poisson's ratio.

Discussion of Test Results

Test results are tabulated in Table 5 for those specimens tested by both static and dynamic methods. Some rather interesting, if not startling, data are shown. The cores of Hypersthene andesite have relatively high values of Young's modulus and show that static test values exceed dynamic test values under all conditions. Usually, and especially for concrete, the dynamic E, sometimes known as the instantaneous or tangent modulus, is considered to be higher than the secant modulus determined by static methods. Certain rocks, notably diorite gneiss and granite, indicated this unusual tendency under some moisture conditions. The andesite sample tested gave very uniform results for Poisson's ratio. Static tests at 75-percent saturation were less uniform than were the dynamic tests at 100 percent saturation. The limestone cores gave excellent agreement between the test methods and few inconsistent values.

Six claystone cores were also included in this tabulation, but were tested in air-dry condition only as no oven-dried or saturated tests were possible because of the nature of claystone. These specimens air slake readily and any oven drying or saturation process would have destroyed them. In general, the tests on these cores were consistent.

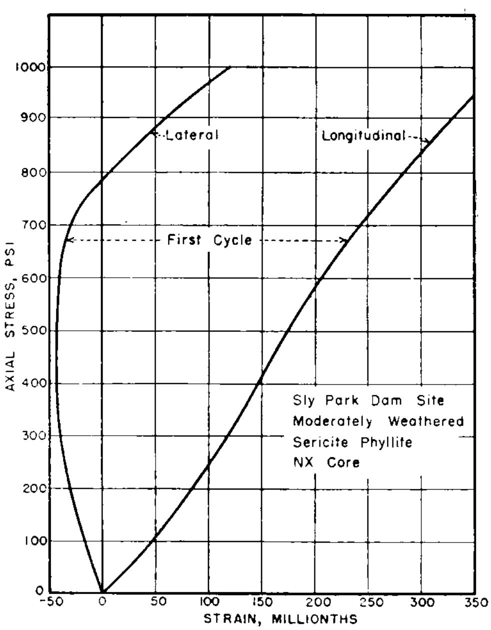


Figure 9. Typical stress-strain curves for phyllite cores.

Schists were, in general, interspersed with mineralized seams of various types. These seams tended to produce distorted vibrations which frequently displaced the node points toward the end of the specimens; and in one instance, the flexural nodes followed a small diagonal seam in the specimen as the core was rotated and tested in the various positions. The seam acted as though it were literally a screw thread moving the nodal points of resonant vibration back and forth along it with one nodal point coinciding approximately to the location where the seam crossed a vertical plane through the test specimen.

Only one of the phyllite specimens gave rational results in dynamic tests, and all values of Poisson's ratio were irrational in the static tests. Figure 9 is typical of data from static tests which show irrational values of Poisson's ratio in that the indicated values usually increase and become positive at higher loads. These static test results show negative Poisson's ratio throughout a large portion of the test. This particular series is unique in that the dynamic tests indicated irrationally high values of Poisson's ratio. The phyllite specimens were difficult to test by the dynamic method, not because they were hard to vibrate, but because many false or combination frequencies were present, and node points were usually either poor or displaced. One specimen showed a partial vibration. That is, a part of the specimen on one side of a small seamy vein vibrated as though it were a separate specimen, while the rest of the sample showed no vibration. Also, one specimen in this group apparently vibrated at flexural mode and torsional mode at the same frequency. The particular mode of vibration depended upon the direction from which the frequency was approached in tuning.

Basalt cores were tested from two different sources, ranging from finely vesicular to a swiss-cheese texture, and gave scattered irrational values of Poisson's ratio. Values which are negative, approximately 0, or plus 0.5 or greater, are considered to be irrational. Discussion of irrational values will be presented later.

Greywacke cores were typical of specimens

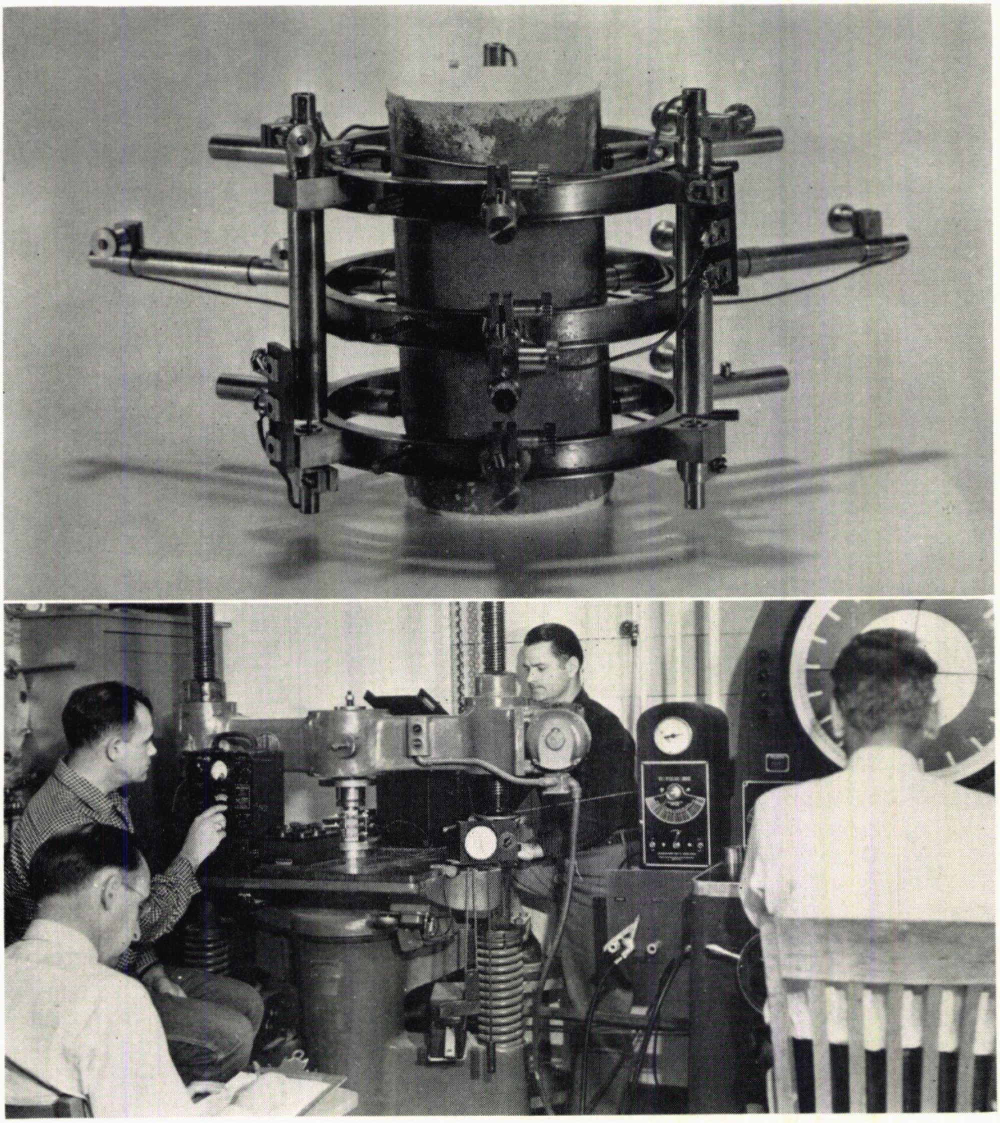


Figure 10. The static strains on an NX core are measured electrically while under load.

indicating irrational values of Poisson's ratio. One group characterized by coarse grained cores gave irrational values throughout all tests. Another group of cores tended to be irregular, especially when saturated, but were irrational only in the static tests. These cores, in general, gave sharp clear indications of both flexural and torsional resonant frequencies.

Sandstone cores had predominantly irrational values of Poisson's ratio and showed damage by oven drying, increased damage by the following saturation, and no recovery. Repeated wetting and drying of several sandstone cores reduced values of elastic properties. Good agreement on Poisson's ratio was obtained between dynamic and static tests. Low amplitudes of resonant vibration and excessive power for driving was required for these specimens, and even then frequent half or double frequencies were noted.

Irrational Values of Indicated Poisson's Ratio

Irrational values of Poisson's ratio occurred throughout these tests in both good and poor specimens and in both dynamic and static tests. It will be noted that irrationalities are more common among specimens of low elasticity or specimens having large differences between dynamic and static test results. The test methods were completely independent from each other, the static tests being performed as shown in Figure 10. Harness frames transmitted deformation from the specimen to precision electric gages. Examination of the data in Table 5 will reveal that usually irrational values in one test were accompanied by irrational, or near irrational, values in the other test. Although irrational values were usually in the form of negative Poisson's ratio, some specimens, notably the phyllite cores, gave impossibly high Poisson's ratio in dynamic tests and negative values in the static test. The incidence of irrational values was not quite so high or the magnitude not quite so large in the static test as in the dynamic test.

Early investigators, contemporaries of Lord Rayleigh, at first thought that Poisson's ratio should be 0.25 for all materials. They later discovered that this was not true, and decided that Poisson's ratio might possibly vary from 0 to plus 0.50, but that values outside of this range were impossible. Lord Rayleigh (*9*, *10*) points out that while negative values of Poisson's ratio may be mathematically possible, they are inconsistent with stability.

The author can see no possible way that a material could have an actual negative Poisson's ratio. However, these tests by both methods are stable in cases that definitely indicate such values. Frequently these values have a high degree of reproducibility in either static loading or sonic resonance tests. The only possible explanation the writer can provide at this time seems to be a far-fetched and improbable set of conditions, namely, that such rocks have a directional characteristic in which residual stresses before core drilling were such that their relief caused a distortion of oriented elements, which actually causes a lateral contraction of the specimen upon application of load. The effect would be somewhat similar to each element acting as a little column and buckling in a predetermined direction. Regardless of how fantastic this may seem, there must be some logical explanation of what happens.

Peculiar Sonic Behavior of Some Igneous Rocks

Certain igneous rocks, especially those having a relatively coarse-grained structure, seem to be mechanically bonded together. Polished surfaces show marks between crystals which may be microscopic separations.

This class of rock, of which granite is probably the best-known example, frequently tax the patience and ingenuity of the operator. The specimens appear to vibrate with many resonant frequencies, but usually, except for deteriorated samples, it is possible to get excellent test results. The net result is an apparent multitude of resonant frequencies, either in flexure or torsion, or both, with little or no indication of nodes, except at true flexural or torsional resonance. The author uses the espression "vibrating like a bag of marbles" to describe the action of the worst of these offenders.

Vibration of Composite or Mixed Specimens

Rock foundation cores occasionally consist of two types of material. When this occurs, one type of material usually dominates and the test results will not be an average but will be similar to those of the dominate material. Dynamic tests of claystone Specimens 16-17 acted like Specimen 17 in the static test while Specimen 16, which was very weak in the static test, apparently had little or no effect on the dynamic test, except to supply length and weight.

CONCLUSIONS

1. Low-frequency, high-amplitude vibration groups have a greater tendency to cross cracks than do high-frequency, low-amplitude groups, even within the audible range.
2. Low-frequency, high-amplitude vibration groups are unsuitable for detecting and measuring cracks by sonic-velocity tests, while high-frequency groups, especially ultrasonic pulses, should be suitable for this purpose.
3. Results of investigations indicate that the velocity test method is not adequate for determining time of set of concrete.
4. Wave velocities measured in fresh concrete, while the mixture is still relatively fluid, appear to be transmitted energy waves rather than sonic waves.
5. Wave velocities measured in loose masses of fine aggregates appear to be energy waves rather than sonic waves.

6. The velocities measured through loose or semifluid masses depend upon particle size and, to some extent, upon wave amplitude and frequency.

7. Indicated velocities in fresh concrete approach a minimum, depending upon the type of signal used.

8. Difficulties in sonic resonance testing of rock cores can frequently be traced to some type of inhomogeneity.

9. Irrational values of Poisson's ratio which were observed in both static and dynamic tests, are probably due to some special type of inhomogeneity or anisotropy rather than errors in the test methods.

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APPENDIX A

Since the original interval timer developed in the reclamation laboratories was not of the type showing the test signals on an oscilloscope screen, it was decided to build another set of equipment with which shorter time intervals could be measured. This method would employ visual observation of the leading edge of the signal trace. The block diagram in Figure A is used to illustrate the operation of the new timerscope circuit. Three types of reactions are started through the electronic circuit instantly when the pendulum hammer strikes the specimen. First, the square-wave generator at the top of the diagram supplies a pulse to the intensifier grid and also to the sweep circuit of the oscilloscope. Second, the square wave generator near the center of Figure A initiates a pulse to the differentiator amplifier where a time delay is incorporated into the circuit making it possible to delay a positive pip on the scope for a calibrated amount of time within the limits of the circuit. Third, the crystal pickup feeds a negative pulse to the mixer amplifier from which it is fed into the scope.

The scope is blank except when an impulse is imparted to the specimen. At that time a positive pip with calibrated delay for measuring purposes is fed into the scope as described above. The crystal pickup feeds an impulse into the mixer amplifier where it is combined with the pulse coming from the differentiator

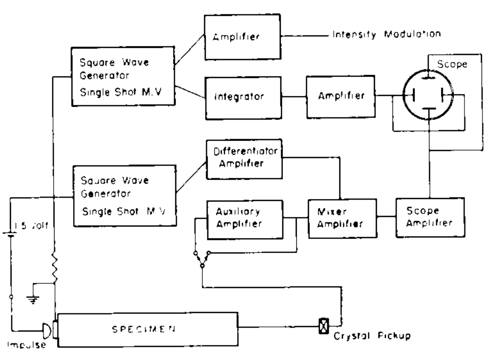

Figure A. Block diagram of timerscope operation.

amplifier, and then fed into the vertical amplifiers of the scope. This impulse from the pickup appears on the scope as a negative pulse so that its initial trace is easy to match with the positive or timing pulse.

All reactions are started at the same instant by an electrical contact on the pendulum hammer. The delay of the signal from the pickup is exactly the travel time of the signal plus pickup delay. This delay is measured by matching the positive target pip from the differentiator amplifier directly above the negative pulse from the pickup and reading the calibrated delay time.

APPENDIX B

If the signals were true sound waves, traveling through a mass of sand and water, then the measured velocities should be between the velocity of sound in rock and the velocity of sound in water. The velocities are actually much slower, indicating that an energy wave must be involved which crosses the spaces between the sand grains at a much slower rate.

If we substitute small uniform spheres for sand in the test form and t_Δ is considered as the time for Sphere 1 to be displaced until it strikes Sphere 2 or Sphere 2 to strike Sphere 3, etc., then the time t_Δ will be found as follows:

$$\text{Displacement} = s = V_0 t + 1/2\, at^2$$
$$\text{Force} = F = Ma$$

as derived from Newton's laws of motion where

V_0 = the initial velocity of a particle (sphere)
a = acceleration
M = mass of the particle (sphere)

If displacement, s, be made equal to Δ and the time required to be t_Δ when

$$V_0 = 0$$
$$\Delta = \tfrac{1}{2}\, at_\Delta^2$$
$$t_\Delta^2 = \frac{2\Delta}{a} \quad \text{and} \quad a = \frac{F}{M} = \frac{Fg}{W}$$

where

W = weight of particle (sphere)
g = earth gravity constant
ρ = the density of the particle (sphere)

then

$$W = \rho \tfrac{4}{3}\pi \left(\frac{d}{2}\right)^3 = \frac{\rho\pi d^3}{6}$$

$$a = \frac{Fg}{\frac{\rho\pi\, d^3}{6}} = \frac{6Fg}{\rho\pi\, d^3}$$

$$t_\Delta^2 = 2\Delta\left(\frac{\rho\pi\, d^3}{6Fg}\right) = \frac{\rho\pi\Delta\, d^3}{3Fg}$$

$$t_\Delta = \sqrt{\frac{\rho\pi\Delta\, d^3}{3Fg}}$$

is actually the thickness of the water film separating the particles and should, in general, be independent of the diameter d.

Let

$$K^2 = \frac{\rho\pi\Delta}{3g}$$

$$t_\Delta = K\sqrt{\frac{d^3}{F}}$$

if

F_0 = the force striking per unit area
F = the force per particle

then

$$F = \frac{F_0}{n^2} = \frac{F_0\, d^2}{L^2} \quad \text{approximately}$$

$$t_\Delta = K\sqrt{\frac{d^3}{\frac{F_0\, d^2}{L^2}}} = K'\sqrt{d}$$

hence

$$\bar{V} = \frac{1}{\frac{1}{V_s} + \frac{t_\Delta}{d}} = \frac{1}{\frac{1}{V_s} + \frac{K'}{\sqrt{d}}}$$

The computed values shown as the dotted line of Figure 8 in the text are based upon this equation using an assumed value of K' = 0.000320 and a test value of V_s = 13,000 feet per second.

DISCUSSION

ERNEST E. MCCOY AND BRYANT MATHER, *Waterways Experiment Station, Corps of Engineers, U. S. Army, Jackson, Mississippi*—Mitchell's discussion of tests of rock cores, and particularly his reports of the irrational values for Poisson's ratio sometimes indicated by the results of such tests, suggests the desirability of recording somewhat similar occurrences encountered in this laboratory and presenting some results of calculations of Poisson's ratio of concrete specimens from dynamic tests.

STATIC LOAD TESTS OF PRISMS OF ROCK

Irrational values of Poisson's ratio were observed among results of several tests made in 1950 on rock specimens. One of two speci-

mens of cherty limestone from northwestern New York, containing numerous fine cracks predominantly in the chert areas, gave a high negative value of Poisson's ratio. Two of four specimens of Sioux quartzite from South Dakota gave negative values. Each specimen was sawed into the form of a prism, all dimensions being between 2 and 4 inches. Measurements were made using SR-4 strain gages with nominal gage length of $^{13}\!/_{16}$ inch. Two gages, one vertical and one horizontal, were used on each of the quartzite specimens, and four gages, two vertical and two horizontal, on the limestone. At 1000-psi. stress (static load), computed values of Poisson's ratio were −0.52 and −0.82 for the two quartzite specimens, and an average of −1.00 on two opposite faces of the limestone specimen. Since there were cracks in this specimen in the immediate vicinity of the gages, it was assumed that the irrational values (−0.07 on one side and −1.93 on the other) were the result of nonrepresentative strain indications. The quartzite specimens, however, were relatively free of defects. An indicated horizontal contraction continued as these specimens were tested up to 9000-psi. compressional stress (the approximate limit of loading). The Poisson's ratio became less negative as the load increased but never attained a positive value.

DYNAMIC TESTS OF CONCRETE BEAMS

Our experience with determinations of Poisson's ratio from dynamic tests has been limited in the main to testing prismatic concrete specimens. A large series of these have been tested in a single research program beginning in 1952. The specimens were 3½ by 4½ by 16 inches in size and were generally without cracks of any kind. They were moist-cured continuously from the time of fabrication to date of test. Pairs of odd-numbered specimens were tested at 14 days of age, and companion pairs of even-numbered specimens at 180 days of age. Poisson's ratio was calculated from resonant flexural and torsional frequencies using the methods given in ASTM Designation C 215-52T. The specimens all contained the same aggregates (crushed limestone coarse and fine, ¾-inch maximum size and a wide variety of portland cements and cementing-medium combinations. There was not a single instance of irrational Poisson's ratio in the results of the 2,432 tests which comprised the program. Results were quite uniform, as illustrated by the following summary:

	Age	
	14 Days	180 Days
Average Poisson's ratio	0.238	0.242
Range (from averages of pairs)	0.13 to 0.30	0.16 to 0.36
Range (absolute)	0.11 to 0.33	0.13 to 0.39
Coef. of variation	11 percent	9 percent

DISCUSSION OF TESTS OF ROCK CORES

Mitchell's data were obtained from tests of relatively small cylindrical rock specimens. These specimens had seams and cracks, and no doubt there were many instances of anisotropic elastic properties owing to orientation of structural and textural features. Although it appears that no positive explanation of the cause of irrational values of Poisson's ratio can be derived at the present time from his data, Mitchell has presented information of much interest. There appears to be no significant correlation between negative values of Poisson's ratio determined by the dynamic and static methods among his data. By noting the position of nodal points, Mitchell apparently has taken the best-possible precaution against obtaining irrelevant frequency readings.

When cores are taken that represent material naturally under a heavy superjacent load, there are at least two stress phenomena present: instantaneous elastic relief of stress and exponential creep. These phenomena, generally regarded as spatially linear, would hardly account for a special reorientation of the structural elements of the rock under most circumstances. However, when a core is drilled, the progressive operation of drilling might permit an annular stress gradient near the edge of the bit of sufficient strength to set up an asymmetrical orientation of structural elements. Mitchell, rather reservedly, suggested that drilling might set up an orientation of the elements of the specimen whereby a negative Poisson's ratio would be obtained. Such an explanation seems as logical as any other that presents itself at the present time. It would possibly be helpful to know whether the Poisson's ratio of such a core as he describes is dependent on time. Consideration might also be given to the analogy of the fore-

ing of a stopper into the neck of a bottle. In this situation, it would be quite possible to measure simultaneously, on the approximately vertical face of the stopper, both vertical and horizontal contraction produced by the combined action of the vertical load and the lateral restraint.

In this connection, attention may also be directed to the large differences that may exist between the properties of core samples and the masses from which they were taken. Fox states:[1] "Test results from core samples usually give values considerably higher than the rock in place and must be used with caution. For most igneous and metamorphic rocks the values obtained from core samples may average as high as 100 percent above field conditions."

The following observation that was made by Faraday in a letter to Tyndall is regarded as pertinent:[2] "The more we can enlarge the number of anomalous facts and consequences, the better it will be for the subject, for they can only remain anomalies to us while we continue in error."

[1] Portland P. Fox, "Determination of the Modulus of Elasticity of Rocks for Pressure Tunnels and Shafts", *Bulletin* Geological Society of America, vol. 64, December 1953, p. 1425.

[2] Quoted by Joel H. Hildebrand in *Discussions of the Faraday Society* No. 15, 1953, p. 9.

Field Substitution of Flyash for a Portion of Cement in Air-Entrained Concrete

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AFTER a series of laboratory tests, Wisconsin undertook a field investigation of the effects of substituting flyash for a portion of the cement in air-entrained concrete. Requirements for the concrete were standard in all respects except that they called for using 20 lb. of flyash with each 75 lb. of cement and air content of 2.5 to 6 percent.

Test sections were placed with the standard paving mixture using a nominal 5.4 sacks of air-entraining cement per cubic yard, the mixture with the flyash substitution, and the latter mixture with air-entraining agent added to restore the air content to nearly that of the standard. Tests included strength and durability tests on field and laboratory mixed concrete, and strength tests on cores taken from the pavement when it was 39 months old.

The results substantiated those obtained in the earlier laboratory tests and showed that substituting flyash for about 20 percent of the cement increased the strength of the concrete at ages beyond 28 days but reduced the air content and the resistance to freezing and thawing. Restoring the air content of the flyash concrete reduced the strength but restored the resistance to freezing and thawing to a large extent.

Strength tests on the cores showed that in the pavement both the concrete with the flyash substitution and reduced air content and the flyash concrete with air content restored were substantially stronger than the standard concrete. Inspection of the pavement at the time the cores were taken showed no adverse effects on any of the concretes after three years of service.

● A SERIES of laboratory tests[1] indicated sufficient merit for the State Highway Commission of Wisconsin to undertake a field investigation of the effects of substituting flyash for a portion of the cement in air-entrained concrete. The field work was carried out on

[1] Reported in Proceedings of the Highway Research Board, Vol. 29, 1949 and Vol. 32, 1953.