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Evening peak-hour traffic on 13th Street, NW., Washington, D. C.

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U. S. DEPARTMENT OF COMMERCE
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Capacities of One-Way and Two-Way Streets with Signals and with Stop Signs

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by ALEXANDER FRENCH [1]
Highway Transport Research Engineer

In recent years there have been reports from several cities of streets with progressive signal control carrying very heavy traffic at comparatively high speed. Attempts to develop comparable systems in other areas have frequently been unsuccessful. This article describes an intensive study of a major street and indicates the high capacities which are attainable. With information from a number of these facilities it should be possible to determine and measure the effects of the significant factors which make such high capacities possible.

The possible capacity of the one-way through street, 40 feet in width, described in this article was found to be 7,000 vehicles during each hour of green signal time. With the light green during one-half of each signal cycle, it is thus possible to carry 3,500 vehicles during each clock hour. Since an ideal combination of conditions, some of which remain to be measured, is evidently necessary for the attainment of these high volumes, there is no assurance that such high capacities can be designed into a city street system. The traffic volumes given in the Highway Capacity Manual are usually attainable under average conditions, and the values reported therein are generally appropriate for design purposes.[2]

All-way stop signs, the general term which includes four-way stop signs as well as the situation when one or more of the streets are one way, have been used frequently during recent years. There has been considerable discussion by engineers as well as the driving public of the relative merits of all-way stop signs, the more usual stop signs on the cross streets only, and signal control. In this article capacities with signal control, with stop signs on the cross street, and with stop signs on all approaches are compared. From this comparison it appears that stop signs on the cross streets only are practical at traffic volumes considerably below those at which the other two types of control are feasible. When both streets are two way, four-way stop signs appear to be practical at traffic volumes approaching those observed for signal control. With the through street one way, however, progressive signal control was found to provide substantially greater capacity than either of the two types of stop-sign control.

[2] *Highway capacity manual*, by the Committee on Highway Capacity, Department of Traffic and Operations, Highway Research Board. Published by the Bureau of Public Roads, 1950.

THE relative merits of each of the various types of control devices have long been a topic of discussion and it is therefore desirable that factual information be obtained for use as a guide in determining the conditions under which traffic signals, two-way stop signs, and four-way stop signs provide the most efficient operation.

Both stop signs and signals are used extensively to control traffic at intersections. Generally, stop-sign control is used only on the less important of the two intersecting streets, but in some instances four-way stop-sign control is used.

This article is concerned with a study of traffic at four intersections while the traffic signals were in normal operation, and also studies at the same locations while two-way stop signs and four-way stop signs were used in place of traffic signals.

Definitions

Progressive signal control.—In the simple progressive system the various signal faces controlling a given street give "go" indications in accordance with a time schedule to permit, as nearly as possible, continuous operation of groups of vehicles along the street at a planned rate of speed, which may vary in different parts of the system.

Possible capacity.—The maximum number of vehicles that can pass a given point on a lane or roadway during one hour under the prevailing roadway and traffic conditions, regardless of their effect in delaying drivers and restricting their freedom to maneuver.

Practical capacity.—The maximum number of vehicles that can pass a given point on a lane or roadway during one hour under the prevailing roadway and traffic conditions, without unreasonable delay or restriction to the drivers' freedom to maneuver.

Cross street stop-sign control.—The cross street approach or approaches to an intersection are controlled by stop signs while the through street is uncontrolled.

All-way stop-sign control.—All intersection approaches are controlled by stop signs. The term "four-way stop sign" is not applicable at intersections where one or both of the intersecting streets are one way.

Loaded cycles.—A cycle was considered to be loaded at an intersection approach if vehicles were traveling through the intersection during the entire green period and the flow was stopped by the amber or red signal.

Description of Study Site

The four intersections studied are on 13th Street, N. W., in Washington, D. C., a four-lane street which carries more than 3,200 vehicles per hour during the peak period. It is operated one way inbound during the morning peak period and one way outbound during the evening peak period, and for the remaining time it operates as a two-way street with parking on one side. The 1.8-mile section of 13th Street between Logan Circle and Spring Road with a peak-hour overall speed of about 18 miles per hour is an unusually efficient arterial street, carrying far heavier traffic than

[1] The data were collected in the field by 19 junior highway engineers. A preliminary report by four of these engineers, Robert D. Bee, Walter W. Bryant, Dwight A. Hodgens, Jr., and Joseph Rekas, was prepared as part of the Bureau of Public Roads Junior Engineer Training Program. This article is based chiefly on further analysis of the data. The District of Columbia Department of Motor Vehicles and Traffic supplied the signs and adjusted signal timing as directed by John H. Mitton, Assistant Director and Traffic Engineer. The Metropolitan Police provided officers for emergency control under the direction of John J. Agnew, Deputy Chief of Police.

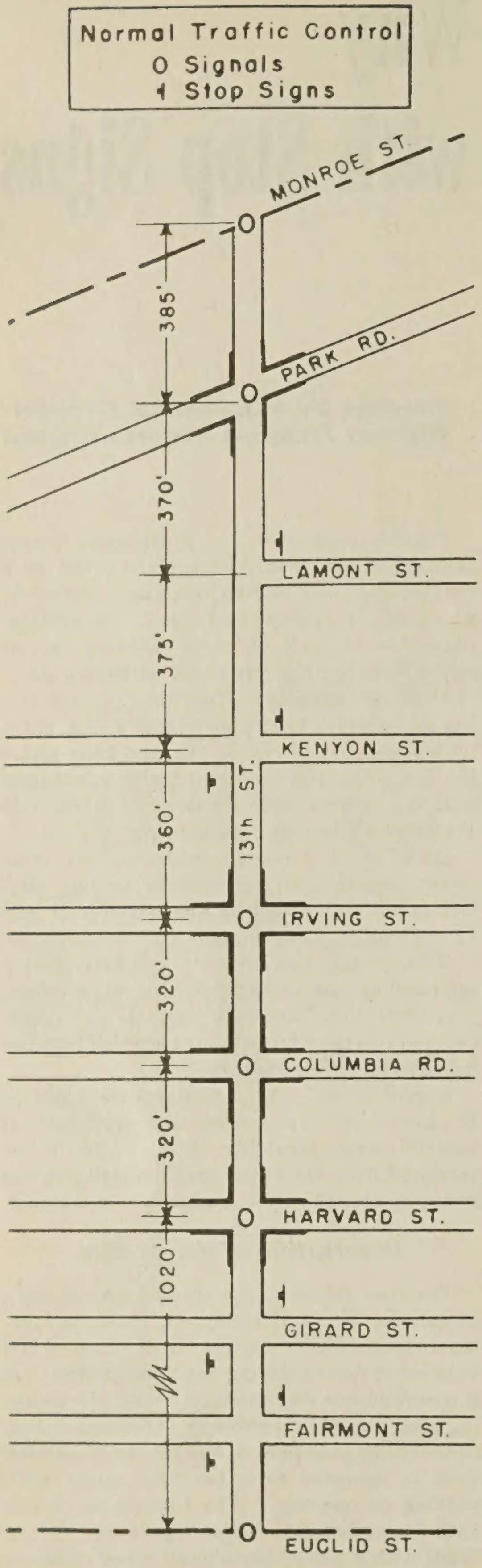

Figure 1.—Location of four intersections on 13th Street included in study.

most streets of this width are capable of accommodating under usual conditions. The three types of intersection control were observed under heavy traffic loads at intersections with both one-way and two-way streets.

Thirteenth Street is a through north-south street carrying much heavier traffic volumes than the cross streets. The cross streets at which studies were made are as follows: Irving Street and Park Road, which carry two-way traffic; Harvard Street, which is one way eastbound; and Columbia Road, which is one way westbound. The general layout of the study area is shown in figure 1. Details of each intersection are shown in figure 2, while figure 3 shows typical traffic on 13th Street when operating two way and one way.

All four intersections are normally controlled by an interconnected system of coordinated fixed-time traffic signals. There is a single dial in the time controller at each intersection. Consequently, the 80-second signal cycle and the stop and go intervals for 13th Street and the cross streets remain constant throughout the day. The signals are interconnected with a master controller, however, so that the time offsets between successive signals on 13th Street can be changed. This provides for progressive traffic movement favoring the desired direction of travel at different times of the day.

Three different signal progressions are used. The first, which is used during the morning rush period, provides progressive movement for 13th Street traffic while it is one way inbound. The second, which is used during nonrush periods, provides reasonably good progression in both directions while 13th Street carries two-way traffic. The third, which is used during the afternoon rush period, provides excellent progression while 13th Street is operating one way outbound.

Accident data for 13th Street are compared with those for other arterial streets and urban freeways in table 1. The accident rate is 13 percent higher than the average based on a nationwide study of representative arterial city streets. This higher accident rate may be due in part to the very complete reporting of accident data in the District of Columbia. It is not believed to indicate a significantly higher accident potential. The accident rate on 13th Street is much higher, however, than that for urban freeways with full control of access although the fatality rates are the same.

Scope of Study

The principal field studies were conducted on three weekday afternoons in March 1954. Operation with signal control was observed the first day. The green time on the cross streets was reduced in an attempt to provide a continual backlog of vehicles on cross streets so that their possible capacities could be determined. On another day, stop signs were used to control traffic on the cross streets. On a third day, stop-sign control was used on all approaches to the intersections. Additional field studies were conducted in November to measure the delay to traffic with the normal signal timing and to determine the capacity of 13th Street with two-way traffic controlled by signals.

Table 1.—Accidents per hundred million vehicle-miles of travel on 13th Street compared with other facilities

	13th Street [1]	Other arterial streets [2]	Urban freeways [2]
Reported accidents	1,091	966	146
Fatalities	2	3	2

[1] Logan Circle to Spring Road, 1.8 miles, Jan. 1, 1952–June 30, 1954.
[2] These are preliminary figures for arterial streets with no access control and for urban freeways with full control of access. Data were developed from cooperative studies made by the Bureau of Public Roads and State highway departments.

Manual counts of all through and turning movements, classified by type of vehicle, were made at all entrances to the intersections. These were recorded for every cycle when signals were in operation, and every 2 minutes when signals were off and traffic was being controlled by stop signs. Fully utilized "go" periods (loaded cycles) were noted while observing traffic controlled by signals. To provide data from which vehicular delay might be computed, counts were made of standing vehicles at each intersection approach at regularly spaced intervals. Intervals of 30 seconds were used when signals were in operation and 2-minute intervals were used when stop-sign control was employed.

Field data were summarized by 10-minute periods. Rates per hour of green signal time were calculated so that the rates are comparable regardless of signal timing. The rate per hour of green was calculated by dividing the hourly volume as normally determined by the percentage that the green period, not including any amber time, was of the total cycle.

To calculate total delay, the number of stopped vehicles as determined by the periodic count was multiplied by the time interval between counts. This value was then divided by the number of vehicles entering the intersection to determine the delay per vehicle.

Summary of Findings

A definite value for either practical or possible capacity was determined for each of the three types of intersection control included in this study. The effect of one-way and two-way operation on the capacities of both through and cross streets and the effect of parking on the through street were also determined. These determinations were based on the magnitude of the delays to traffic as well as on the traffic volumes observed. The studies were conducted at intersections where the through street is 40 feet wide and the cross streets 30 feet wide with parking on one side. The intersections are in a densely developed residential area with comparatively few pedestrians. Traffic on the through street was exceedingly heavy during the rush periods, and on each of the four important cross streets it was seldom less than 250 vehicles per hour during the day. The following are the more important findings for the prevailing roadway and traffic conditions:

Progressive signal control

The capacity of 13th Street with one-way operation and parking prohibited was found to be 1,750 vehicles per hour of green per 10 feet of street width.

The capacity of 13th Street when operating two way with parking on one side was 37

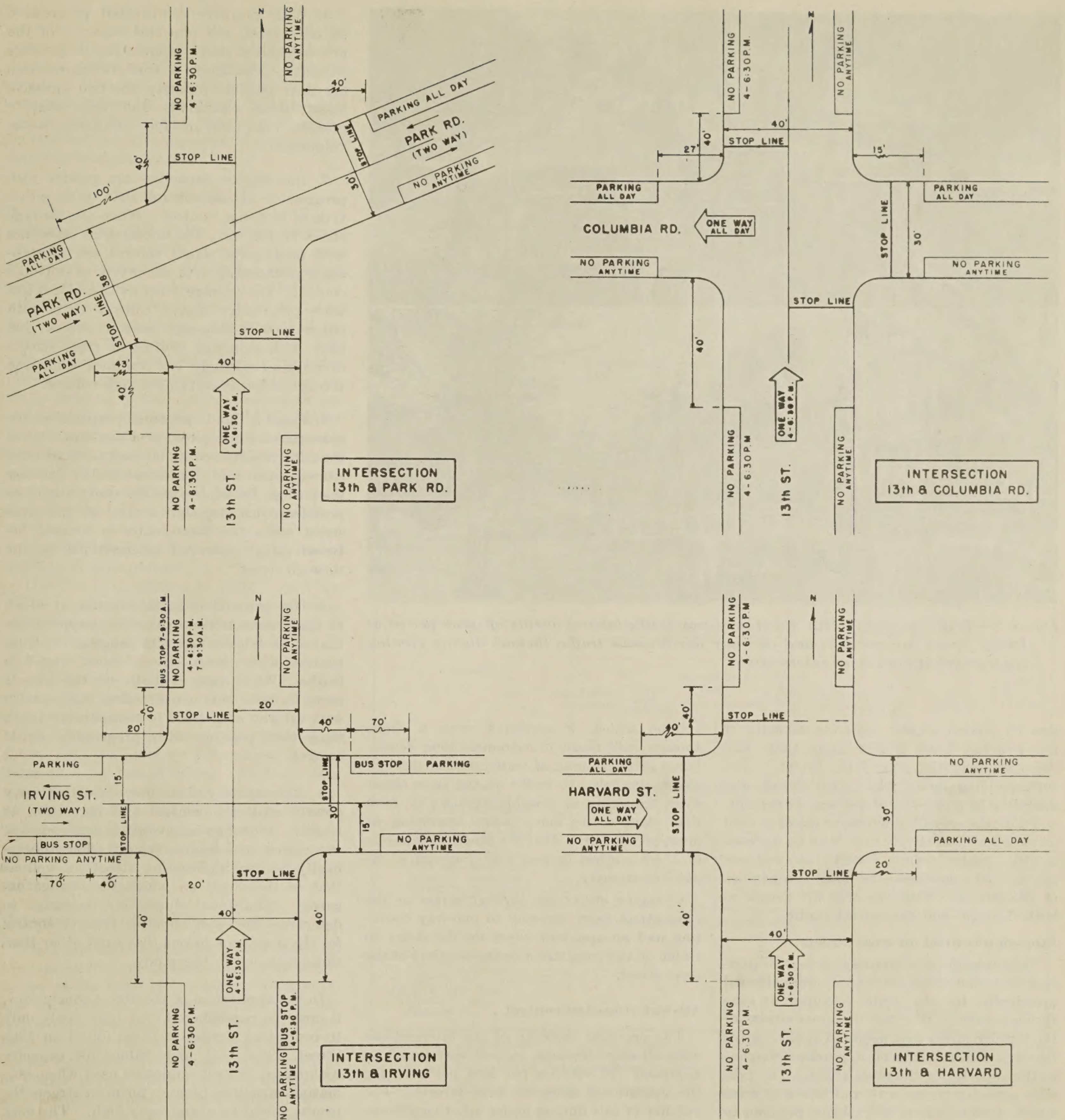

Figure 2.—Detailed information for intersections studied.

percent greater at two-way cross streets and 48 percent greater at one-way cross streets than with parking on both sides. With no parking on either side, the through-street capacity was 75 percent greater at two-way cross streets and 97 percent greater at one-way cross streets than when parking was permitted on both sides.

Changing from two-way to one-way operation on the through street, with no parking, increased the capacity of the through street 25 percent at intersections with two-way cross streets and 11 percent at intersections with one-way cross streets.

The capacity of the through street was increased 119 percent by the elimination of parking from both sides and by the use of one-way rather than two-way operation.

Changing from two-way to one-way operation and eliminating parking on the through street increased the capacity of the two-way cross streets 11 percent and the one-way cross streets 20 percent.

The capacity of the one-way cross streets

Figure 3.—Traffic scenes on 13th Street: two-way traffic (above) during off-peak period at Irving Street intersection, and one-way northbound traffic (below) during evening peak period at Park Road intersection.

was 15 percent greater than the capacity of the two-way cross streets while there was two-way operation on 13th Street. The corresponding figure while 13th Street was operating as a one-way street was 24 percent.

Traffic delays with progressive signal control increased comparatively little with an increase in the traffic volume until the volumes approached very closely the possible capacities of the streets. This was true for delays to both through- and cross-street traffic.

Stop-sign control on cross streets

The capacities of the cross streets with stop-sign control on these streets only were affected principally by the traffic volume on the through street. Whether the cross streets or the through street were operated as one-way or two-way streets made no apparent difference in the capacity of the cross street. The possible capacity of one-way and two-way cross streets decreased from 800 vehicles per hour to 400 vehicles per hour when the volume on the through street increased from 1,300 to 2,500 vehicles per hour. The decrease in capacity of the cross streets was not, however, directly proportional to the increase in volume on the through street.

Delay to traffic on the through street was practically nil, regardless of the traffic volume on the cross streets.

The delay to traffic on the cross streets was the principal criterion for determining possible and practical capacities with stop signs on the cross streets. The possible capacity is the volume which, if exceeded even a slight amount, will result in extremely long delays. For a given volume of traffic on the through street, the delay to traffic on the cross street when operating at possible capacity is more than double the delay when operating at practical capacity. Also, the practical capacity of a cross street is less than one-half of its possible capacity.

Changing either the through street or the cross street from two-way to one-way operation had no apparent effect on the delay to traffic on the cross street or the capacity of the cross street.

All-way stop-sign control

The practical capacity of the intersections with all-way stop-sign control was found to approach 500 vehicles per lane per hour for the operational lanes on both streets. The validity of this finding under other conditions with a high traffic volume per lane on one of the streets and a low volume per lane on the other street was not determined.

With similar traffic volumes per lane on both streets, the delay per vehicle on the through street was approximately the same as the delay per vehicle on the cross street.

Conclusions

The following conclusions appear to be justified for the conditions under which these studies were conducted:

1. With properly coordinated progressive signal control, the practical capacity of the one-way street closely approaches its possible capacity. The delays to traffic when volumes are near possible capacity are not excessive under these conditions and only slightly greater than the delays at much lower volumes.

2. Intersection capacities are greater with progressive signal control than with either type of stop-sign control. When the through street is one way, the intersection capacities with progressive signal control are substantially greater than with either type of stop-sign control. The average delay to all traffic is less with progressive signal control than with either type of stop-sign control, except possibly with stop-sign control at cross streets carrying exceedingly low volumes while the through street is carrying a high volume.

3. Possible and practical capacities for cross-street stop-sign control are much lower than for progressive signal control or all-way stop-sign control. Somewhat higher volumes than those found during this study might be practical with stop-sign control on the cross street when the intersection is located between signal-controlled intersections on the through street.

4. The capacity of an intersection at which all traffic is controlled by stop signs approaches that of an intersection with progressive signal control when both streets carry two-way traffic. When one or both of the streets carry traffic in only one direction, the capacity with all-way stop signs is considerably lower than that possible with progressive signal control.

5. The capacity of an intersection will vary greatly with the control and regulation of traffic. A change in operating conditions at one intersection approach may affect the capacity of the intersecting street as well as that of the street on which the change occurred. Additional studies are necessary to determine the most effective type of control for the many conditions that exist other than those included in this study.

In the application of the above conclusions, it must be remembered that they apply only to conditions similar to those found on 13th Street. The very high values for capacity with signal control cannot be used when estimating capacities because for most streets the results would be erroneously high. The very efficient operation of the progressive signal system is emphasized since it expedites the movement of exceedingly high volumes of traffic to a degree seldom equaled on streets of this type. It may be significant that most of the drivers on 13th Street during the peak period use this street daily and are therefore practiced in maintaining optimum speed and spacing.

These facts are important considerations when comparing the results of this study with those of other studies which do not show the same advantages for signal control as compared with stop-sign control.[3] There is close agreement, however, in the results of this study and other studies in the traffic volumes accommodated by the two types of stop-sign control. The fact that 13th Street can accommodate smoothly such high traffic volumes at a reasonable speed for urban conditions is a tribute to the traffic engineers who operate the progressive signal system.

Signal Control

Despite the high traffic volumes observed at the four intersections while they were controlled by signals, the traffic demand on some of the approaches was insufficient to utilize fully the green or "go" interval during any of the signal cycles. On several of the approaches the green interval was fully utilized during a few cycles only. The green interval was considered fully utilized when the traffic demand at the observed approach was equal to the possible capacity of that intersection approach with vehicles continuously entering the intersection throughout the green interval. Cycles during which these conditions occurred are called loaded cycles. A loaded cycle for an intersection approach is independent of the traffic on the other approaches which may or may not be loaded during the same period.

The first four columns of table 2 show the traffic volume on 13th Street, the percentage of dual-tired vehicles, and the percentage of right and left turns during the 1-hour period of maximum traffic. The next column shows the number of seconds of green signal time per cycle. The traffic volume per hour of green signal time follows. The highest hourly rate observed during 10 consecutive minutes is shown next, followed by the rate per hour of green time for loaded cycles. All these data were compared for similar intersection approaches, and the estimated possible capacity was determined for each approach. This is tabulated in the last column of the table. Table 3 is similar to table 2 and lists the data for the cross streets.

Table 2.—Traffic data and estimated possible capacity for 13th Street at four signal-controlled intersections

Location	Conditions during continuous 1-hour periods of maximum traffic					Maximum observed traffic volumes per hour of green time during—			Estimated possible capacity per hour of green
	Traffic volume	Dual-tired vehicles	Turning movements: Left	Turning movements: Right	Green time for 80-second cycle	1-hour period	10-minute period	Loaded cycles [1]	
13th Street Carrying Two-Way Traffic [2]									
13th Street, southbound:	*V. p. h.*	*Percent*	*Percent*	*Percent*	*Seconds*	*Vehicles*	*Vehicles*	*Vehicles*	*Vehicles*
At Park Rd., two way	482	3.5	0.4	7.0	38	1,015	2,080	1,530 (1–1)	1,600
At Irving St., two way	524	2.9	1.9	1.3	(3)	1,435	1,720	1,605 (25–39)	1,600
At Columbia Rd., one way	557	3.4	-----	6.8	45	990	1,181	None	1,600
At Harvard St., one way	555	4.3	5.4	-----	29	1,531	1,707	1,610 (55–78)	1,600
13th Street, northbound:									
At Park Rd., two way	836	2.7	4.1	6.6	40	1,672	2,124	2,460 (3–3)	2,600
At Irving St., two way	748	2.9	4.3	1.3	(3)	2,100	2,916	2,635 (10–25)	3,000
At Columbia Rd., one way	671	3.3	6.8	-----	45	1,193	1,611	None	3,000
At Harvard St., one way	849	2.8	-----	4.1	29	2,342	3,321	3,576 (5–20)	3,300
13th Street Carrying One-Way Traffic [2]									
13th Street, northbound:									
At Park Rd., two way	2,747	0.9	6.4	10.3	40	5,494	7,140	6,857 (8–16)	7,000
At Irving St., two way	3,128	1.0	1.8	.5	52	4,811	5,623	None	7,000
At Columbia Rd., one way	3,168	1.0	3.6	-----	45	5,633	6,614	None	7,000
At Harvard St., one way	2,987	.8	-----	4.7	50	4,779	5,309	5,637 (4–16)	7,000

[1] The number of loaded cycles observed is shown by the first of the two figures in parentheses; the second figure is the number of consecutive cycles in the period during which the loaded cycles were observed.
[2] Parking is permitted only on the west side used by southbound traffic while 13th Street is operating two way. All parking is prohibited on 13th Street during one way operation.
[3] Twenty-four, 32-second green periods; eleven, 28-second green periods; and ten, 24-second green periods.

Table 3.—Traffic data and estimated possible capacity of four signal-controlled streets crossing 13th Street [1]

Location	Conditions during continuous 1-hour periods of maximum traffic					Maximum observed traffic volumes per hour of green time during—			Estimated possible capacity per hour of green
	Traffic volume	Dual-tired vehicles	Turning movements: Left	Turning movements: Right	Green time for 80-second cycle	1-hour period	10-minute period	Loaded cycles [2]	
Cross Streets Carrying Two-Way Traffic									
13th Street, two way:	*V. p. h.*	*Percent*	*Percent*	*Percent*	*Seconds*	*Vehicles*	*Vehicles*	*Vehicles*	*Vehicles*
Park Rd., eastbound	273	8.0	2.9	9.9	18	1,213	1,502	1,333 (24–40)	1,400
Park Rd., westbound	294	3.4	25.8	1.7	18	1,307	1,551	1,400 (30–45)	1,400
Irving St., eastbound	203	13.8	13.8	18.7	18	902	1,200	1,120 (10–44)	1,200
Irving St., westbound	115	15.6	6.1	7.8	18	511	649	800 (1–1)	1,200
13th Street, one way:									
Park Rd., eastbound	366	2.7	13.1	--------	18	1,627	1,849	1,542 (43–52)	1,600
Park Rd., westbound	304	3.0	--------	4.9	24	1,013	1,249	1,433 (6–52)	1,400
Irving St., eastbound	245	9.8	19.6	--------	18	1,089	1,450	1,055 (40–92)	1,400
Irving St., westbound	179	7.3	--------	16.8	18	795	1,000	850 (8–39)	1,400
Cross Streets Carrying One-Way Traffic									
13th Street, two way:									
Columbia Rd.	465	3.9	7.3	9.0	15	2,480	2,917	2,824 (13–28)	3,000
Harvard St.	447	5.6	20.4	5.1	15	2,384	2,821	3,120 (5–7)	3,000
13th Street, one way:									
Columbia Rd.	668	1.0	--------	12.6	(3)	3,185	3,960	3,553 (41–95)	3,600
Harvard St.	708	1.8	33.5	--------	17	3,332	3,936	3,600 (60–89)	3,600

[1] Cross streets are 30 feet wide with parking permitted on one side except for the west leg of the Park Road intersection. The latter is 38 feet wide with parking permitted on both sides.
[2] The number of loaded cycles observed is shown by the first of the two figures in parentheses; the second figure is the number of consecutive cycles in the period during which the loaded cycles were observed.
[3] Thirty-seven, 15-second green periods; and eight, 25-second green periods.

Determination of possible capacity

The estimated possible capacities shown in the last column of tables 2 and 3 were determined on the basis of the 1-hour, 10-minute, and loaded-cycle volumes. Data for similar intersection approaches were considered collectively in arriving at these capacities. The values represent the possible capacities over a 1-hour period even though volumes for shorter time periods were used in their determination. A detailed description of the method of determining these values is necessary for an appreciation of their reliability.

First, the 1-hour volume was compared with the maximum 10-minute volume and the volume during loaded cycles at the particular approach. The relation between these values as well as the frequency of loaded cycles formed a basis for tentatively estimating possible capacity. For instance, if the 1-hour volume and loaded-cycle volume were well below the maximum 10-minute volume, it was evident that the first two were well below capacity. The presence of only a few loaded cycles during a large number of consecutive cycles would confirm this. In such a case it is possible that even the maximum 10-minute volume was below possible capacity. On the other hand, if the three volumes were about equal and a large and fairly concentrated number of loaded cycles were observed, this was an indication that possible capacity was reached. In such instances a volume somewhat below the maximum 10-minute volume, but not less than the loaded cycle volume, might be tentatively selected as the possible capacity.

After the tentative values for all similar

[3] *A capacity relationship between four-way stop intersection control and fixed-time traffic signal,* by James Madison Hunnicutt, Jr. Thesis, Bureau of Highway Traffic, Yale University, 1954; also *A comparison of delay to vehicles crossing urban intersections, four-way-stop vs. semi-traffic-actuated signal control,* by Edward M. Hall. Student Research Report No. 4, The Institute of Transportation and Traffic Engineering, University of California, Jan. 1952.

intersection approaches were selected by this process they were compared with each other for consistency and reasonableness. In this comparison differences in turning movements, frequency of commercial vehicles, bus stops, and other factors known to affect traffic operation were considered. If the tentative possible capacity estimate for any approach appeared inconsistent, a reappraisal was made. In the case of similar approaches where the tentative estimates were close, a single value was selected for the possible capacity of all. If only one of several otherwise similar approaches was observed at or near possible capacity conditions, the value determined for this approach was assigned to the others.

Unusually high values were determined for the possible capacity of 13th Street when it was operating one way northbound. The highest volumes per hour of green were observed at Park Road where the green time was least. Even here, with only one-half the total cycle green to 13th Street, only eight loaded cycles occurred. With a volume during these cycles of 6,857 vehicles per hour of green and a maximum 10-minute volume of 7,140 vehicles per hour of green, the indicated 7,000 vehicles per hour of green is evidently the approximate possible capacity. Since the turning movements were less at the other approaches, it must be inferred that possible capacities at these approaches are at least equal to the 7,000 vehicles per hour of green determined at Park Road. In the absence of factual data indicating higher capacity, this value was accepted for all four locations, rather than some higher value.

The results of an analysis of the data in tables 2 and 3 indicate that within the range of values observed, the percentage of commercial vehicles and the percentage of left and right turning traffic at the intersections had little apparent effect on capacities of the approaches. An exception is the case of Irving Street which carried about twice as high a percentage of dual-tired vehicles as the other cross streets. It had a capacity somewhat lower than the other cross streets.

The rates of traffic flow based on the 10-minute periods of maximum volume were frequently greater than the rates based on the loaded cycles. These 10-minute periods included many cycles that were not fully loaded. It is evident, therefore, that greater volumes are sometimes carried during unloaded cycles than during loaded cycles. A cycle was considered to be loaded at an interesction approach if vehicles were traveling through the intersection during the entire green signal period and the stream was interrupted by the red signal. Frequently, when all drivers were alert, accelerated quickly, and allowed a minimum headway, all vehicles waiting at the intersection and those arriving during the green interval were able to clear the intersection before the green time expired. A cycle of this type would not be classed as a loaded cycle. At other times one or two slow drivers or some other impediment tended to reduce the movement of vehicles through the intersection and, as a result, the cycle was classified as being fully loaded despite the fact that the number of vehicles counted was less than during other cycles which were not so classified.

Capacities unusually high

The estimated possible capacity of 13th Street operating one way northbound is 7,000 vehicles per hour of green. This is equivalent to 1,750 vehicles per hour of green per 10 feet of width or per lane. This extremely high volume is only 2 percent below the maximum 10-minute volume observed on 13th Street at Park Road. At the time there was little delay to traffic, although some vehicles required more than one signal cycle to clear the intersection. Periodic traffic counts obtained by automatic recorders indicate that traffic volumes of 3,200 vehicles during 1 hour occur frequently. This is equivalent to 6,400 vehicles per hour of green at Park Road. Thus 13th Street often carries a volume for a 1-hour period which is in excess of the peak-hour volume recorded during this study.

The estimated capacity of 3,600 vehicles per hour of green for each of the one-way streets crossing 13th is also unusually high. This is equivalent to 1,200 vehicles per hour of green per 10 feet of surface width,

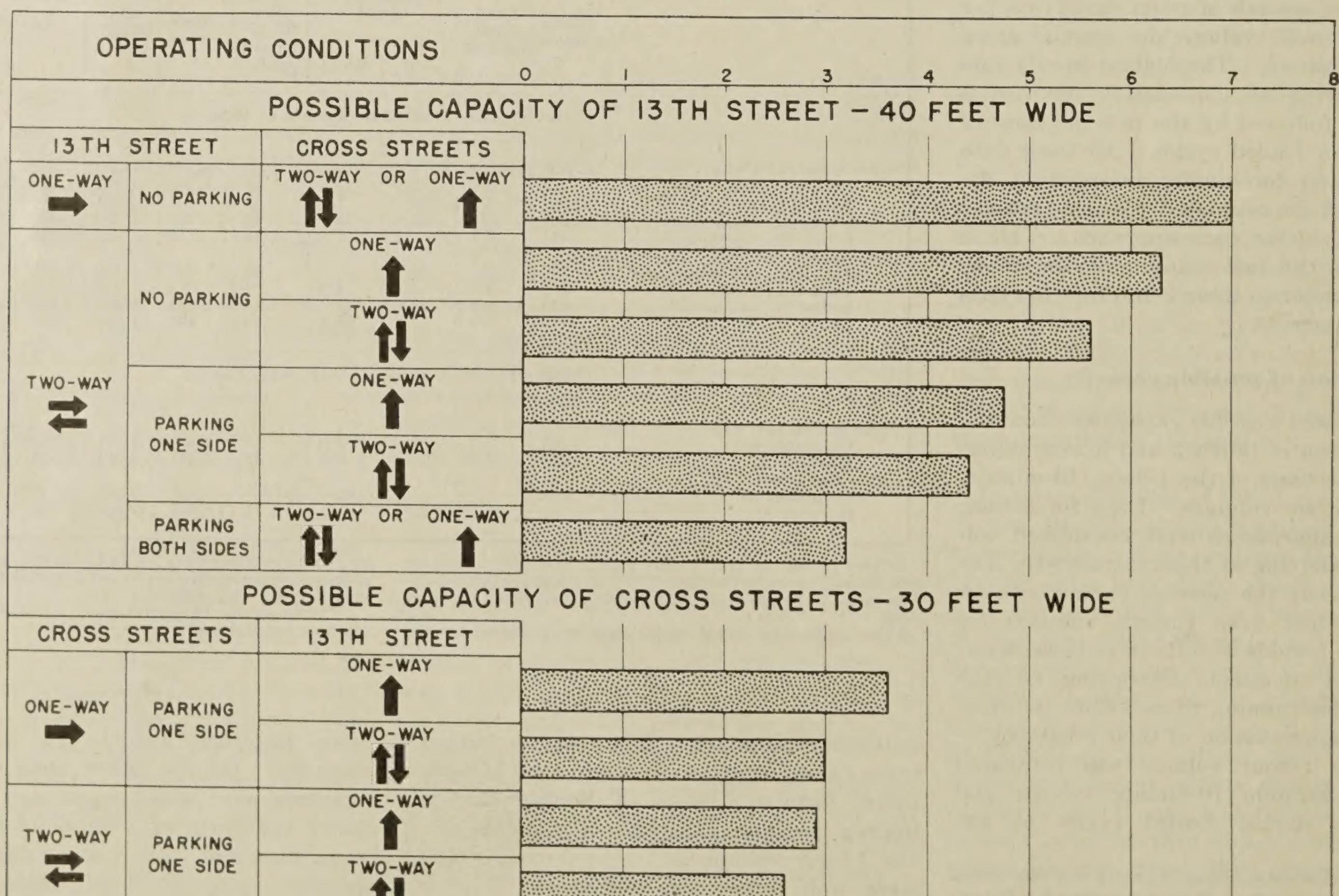

Figure 4.—Street capacity with signal control related to directional operation and parking conditions.

curb to curb, with parking on one side. The capacity of each of the two-way streets crossing 13th Street is also unusually high.

These very high capacities certainly exceed those attained on most city streets. At these high capacities the flow of traffic through an intersection is likely to be greatly affected by a slight change in signal timing, especially in the time offset between succeeding signals which is very critical in a progressive system. Minor accidents, stalled vehicles, and severe weather conditions also result in sharp reductions in capacity. Capacity values considerably lower than those found on 13th Street must therefore be used for design when planning one-way street systems and other improvements to city traffic facilities, or it is likely that adequate capacity will not be provided.

Directional operation and parking related to capacity

By comparing the possible capacities of the approaches, the effect that two-way and one-way operation has on capacity was determined for both 13th Street and the cross streets. The effect of parking on the capacity of 13th Street while operating as a two-way street was also determined by comparing the capacity in the northbound direction with the capacity in the southbound direction. Southbound vehicles could park on the west side of the street while parking was prohibited on the east side. The effects of these conditions are shown graphically in figure 4.

Figure 4 shows that under one set of conditions the possible capacity of 13th Street is only 3,200 vehicles per hour of green signal time, whereas under another set of conditions the possible capacity is 7,000 vehicles per hour of green or 2.19 times as high as the former. The improvement in capacity was realized by eliminating parking and changing from two-way to one-way operation on 13th Street. Likewise, the capacity of a cross street in one case is 2,600 vehicles per hour of green time and in another case, 3,600 vehicles per hour of green. The difference of 1,000 vehicles per hour or 38 percent is a direct result of changing from two-way to one-way operation on both of the intersecting streets and does not involve a change in the parking condition on the cross street.

One of the most important results of the study is illustrated by figure 4, which shows that the operating conditions on one street affect not only the capacity of that street but also the capacity of the intersecting street. Under certain conditions, for example, the capacity of 13th Street is higher when the cross street carries one-way traffic than when it carries two-way traffic. The same is true for the capacity of the cross streets in relation to the directional flow on 13th Street.

Table 4 shows the effect on intersection capacity of each change that was made on 13th Street and the cross streets with respect to directional operation and parking conditions. In this table, 13th Street is referred to as the major street and the cross streets are referred to as minor streets. By using this terminology the results can be more directly compared with the results of studies at other locations and

Table 4.—Percentage increase in capacities of streets at signalized intersections after changing from two-way to one-way operation and after eliminating parking on the major street [1]

Change in operating conditions	Percentage increase in possible capacity for— Major street	Minor street
Changing from Two-Way to One-Way Operation		
Major street, one way with no parking:		
Minor street, two way	25	12
Minor street, one way	11	20
Minor street, one way:		
Major street, two way with parking on both sides	0	15
Major street, two way with parking on one side	8	15
Major street, two way with no parking	12	15
Major street, one way with no parking	0	24
Eliminating Parking on Major Street [2]		
From one side:		
Minor street, one way	48	----
Minor street, two way	37	----
From both sides:		
Minor street, one way	97	----
Minor street, two way	75	----

[1] Parking permitted on one side of minor street in all cases.
[2] Two-way traffic on major street in all cases.

more readily applied to similar situations elsewhere.

It is of particular interest that a change from two-way to one-way operation of traffic on the major street increased the capacity of the minor street as well as the capacity of the major street. The increase for the major street was 25 percent at locations where the minor street was two way and only 11 percent where the minor street was one way. This same change also increased the capacity of the one-way minor streets by 20 percent and the capacity of the two-way minor streets by 12 percent. Thus, the minor streets that were benefited most were those at locations where the major street benefited least by a change in its directional operation.

The cross streets with one-way traffic had a greater capacity than those with two-way traffic. The difference in capacity was greater (24 percent compared with 15 percent) when the major street was also one way rather than two way. This difference in operation did not, however, benefit the capacity of the major street when it carried two-way traffic with parking on both sides or one-way traffic with no parking. One-way operation on the cross streets was of some benefit to the major street when it carried two-way traffic and parking was eliminated from one or both sides.

Eliminating parking on the major street had a far greater effect on its capacity than changing from two-way to one-way operation. The elimination of parking from both sides of the major street nearly doubled its capacity—an increase of 97 percent at intersections with one-way minor streets. At intersections with two-way minor streets, the increase was 75 percent. Eliminating parking on one side of the major street had about half the effect of eliminating parking on both sides.

It should also be pointed out that progressive movement, which reduces travel time, is usually more easily attained on a one-way than on a two-way street. This is an additional and very important advantage of one-way operation.

Stop-Sign Control on Cross Streets

All cross streets were operating at their possible capacities during the rush period on the day that stop signs were used only on the

Table 5.—Traffic volumes and delay per vehicle with stop-sign control at approaches to 13th Street and no control on 13th Street

Location	Period of study [1]	Average for study: Traffic volume, Cross street	Average for study: Traffic volume, 13th Street	Average for study: Delay per vehicle on cross street	Peak 10-minute period: Traffic volume, Cross street	Peak 10-minute period: Traffic volume, 13th Street	Peak 10-minute period: Delay per vehicle on cross street
Cross Streets Carrying Two-Way Traffic							
	Minutes	*V.p.h.*	*V.p.h.*	*Minutes*	*V.p.h.*	*V.p.h.*	*Minutes*
13th Street, two way:							
Park Rd., eastbound	32	272	1,140	0.34	336	1,128	0.43
Park Rd., westbound	32	248	1,140	.48	276	1,140	.61
Irving St., eastbound	38	207	1,265	.41	234	1,236	.15
Irving St., westbound	38	111	1,265	.46	114	1,236	.74
13th Street, one way:							
Park Rd., eastbound	60	322	1,658	2.89	366	1,950	3.21
Park Rd., westbound	60	228	1,658	.51	276	1,746	.74
Irving St., eastbound	60	245	2,172	1.67	318	3,126	1.25
Irving St., westbound	60	143	1,927	.80	168	2,490	.71
Cross Streets Carrying One-Way Traffic							
13th Street, two way:							
Columbia Rd	40	491	1,270	0.59	570	1,260	0.63
Harvard St	34	434	1,200	.63	462	1,152	.70
13th Street, one way:							
Columbia Rd	60	572	1,915	3.81	738	1,866	4.78
Harvard St	60	744	1,785	.92	882	2,070	.64

[1] Where studies exceeded 1 hour, data for the 1-hour period of maximum traffic flow were used. Studies of less than an hour were for the periods immediately preceding the change to one-way operation on 13th Street.

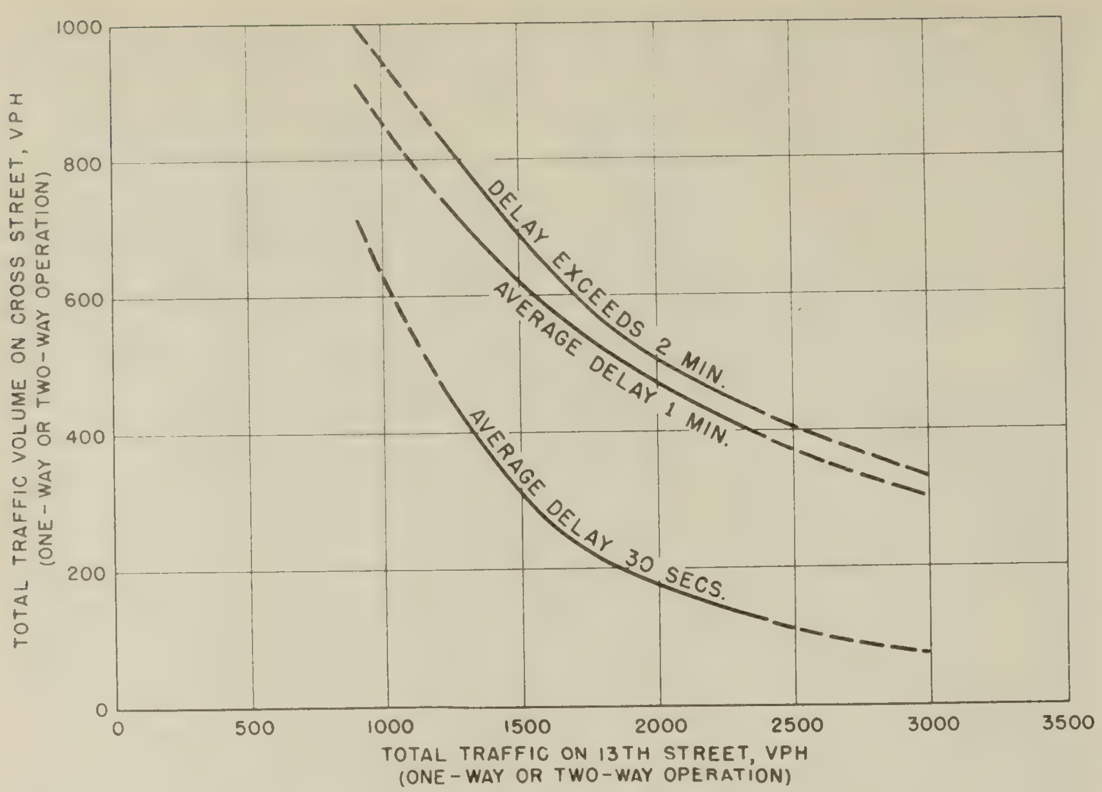

Figure 5.—Delay to traffic on cross streets controlled by stop signs and with no control on 13th Street.

cross streets. Long queues of waiting vehicles developed, and occasionally it was necessary for a police officer to clear the backup on the cross streets. Vehicles on the cross streets were aided somewhat by the signals at intersections on the through street north and south of the study area which caused gaps in traffic on 13th Street.

Table 5 shows the observed traffic volumes and the delays to traffic on the cross streets at the intersections studied. Volumes and delays are listed for the 1-hour and for the 10-minute periods of heaviest cross-street traffic while 13th Street was one way and also while it was two way.

Similar information was obtained for all 10-minute periods during the studies, and the average delays to cross-street traffic were compared for various traffic volumes on the cross streets and on 13th Street. As was expected, the delay to cross-street traffic was found to increase with an increase in the total traffic volume on 13th Street as well as with an increase in the total cross-street traffic. A less expected finding was that the delay to cross-street traffic was independent of the directional usage of either 13th Street or the cross street. Consequently, the delay to cross-street traffic when controlled by stop signs can be shown on one graph representing both two-way and one-way operation on both of the intersecting streets. This has been done in figure 5.

The two lower curves of figure 5 show the combinations of sustained through-street and stop-street traffic volumes which produced average delays to cross-street traffic of 30 seconds and of 1 minute per vehicle. The total traffic volume on 13th Street is shown on the horizontal axis, and the total traffic volume on the cross street is shown on the vertical axis. For example, a cross flow of about 600 vehicles per hour can be accommodated with an average delay of 1 minute when the total traffic volume on 13th Street is 1,600 vehicles per hour. Combinations of volumes which lasted for only short periods sometimes caused delays much longer or shorter than those indicated by the curves. If the volumes were sustained for 20 or 30 minutes, however, the delays were as indicated by the curves.

The third curve in figure 5 shows the maximum volumes that the cross streets can accommodate with various volumes on 13th Street. The delay accompanying these volumes was at least 2 minutes and for any given combination of volumes might have been several minutes. Once the traffic volumes on the cross street and on 13th Street became sufficiently great to cause a delay of 2 minutes, the delay could increase to several minutes within a short period of time with no change in the traffic volumes. This curve therefore represents the traffic volumes for all delays above 2 minutes. It thus represents the possible capacities of the cross streets when controlled by stop signs and with no control on 13th Street.

Volumes which cause a 30-second delay can be exceeded by as much as 100 percent without increasing the delay to more than 1 minute. Volumes which result in a 1-minute average delay, however, can only be exceeded by about 10 percent without increasing the delay to more than 2 minutes.

The small difference between the volumes shown for the curve representing a 1-minute delay and the curve representing a delay in excess of 2 minutes indicates that at these intersections the critical volumes are those producing a delay of between 1 and 2 minutes per vehicle to cross-street traffic.

The volume combinations indicated by the 30-second delay curve fit the usual definition of practical capacity for the prevailing roadway and traffic conditions since greater delay and restriction to movement would appear unreasonable to most drivers. With the normal setting of the signals, average delays of 30 seconds or more to minor-street traffic were infrequent even at the highest volumes observed. In another study it was found that drivers are unwilling to accept longer delays at stop signs than at signals.[4] It is concluded that the practical capacity of the cross street when controlled by stop signs with no control on the through street is represented by the curve for an average delay of 30 seconds.

Stop signs compared with signals

Figure 6 shows the possible and practical capacities of the cross streets with stop-sign control on these streets and the possible capacities with signal control. The curves for stop-sign control represent both one-way and two-way operation and are based on the delay to cross-street traffic. With stop signs on the cross streets, 13th Street could undoubtedly carry as much traffic as with signals but not without an unreasonable delay to cross-street traffic.

For the signals, separate curves are shown for one-way and two-way operation, but separate curves are not shown for possible and practical capacities. With the progressive signal control there was no appreciable increase in the delay to traffic as the traffic increased from comparatively low volumes to those at possible capacity. Thus it was not feasible to establish a value for practical capacity with progressive signal control on the basis of delay to traffic. For the purpose of comparing the three different types of control on the basis of a tolerable delay to traffic, the curves for practical capacity with stop-sign control should be compared directly with the possible capacity curves for signal control.

It may be noted that the curves for signal control intersect the X axis and the Y axis at values below those given in the last column of tables 2 and 3. This is because the amber time in each cycle was not included in the green time when calculating the traffic volumes for the signal-control curves.

From figure 6 it may be seen that both the possible and the practical capacities of the cross streets are much lower with stop-sign control than with signal control. This is especially true with one-way operation on both streets. Even with volumes as low as 1,000 vehicles per hour on 13th Street and with two-way traffic on both streets, the possible capacity of the cross streets with stop signs was about one-half the capacity with signal control. With one-way traffic and at other volumes on 13th Street, the difference was even greater.

[4] *Effects of reversible lane movement signalization of three-lane highways*, by M. Mansfield Todd. Proceedings of the Highway Research Board, vol. 30, 1950, pp. 346-354.

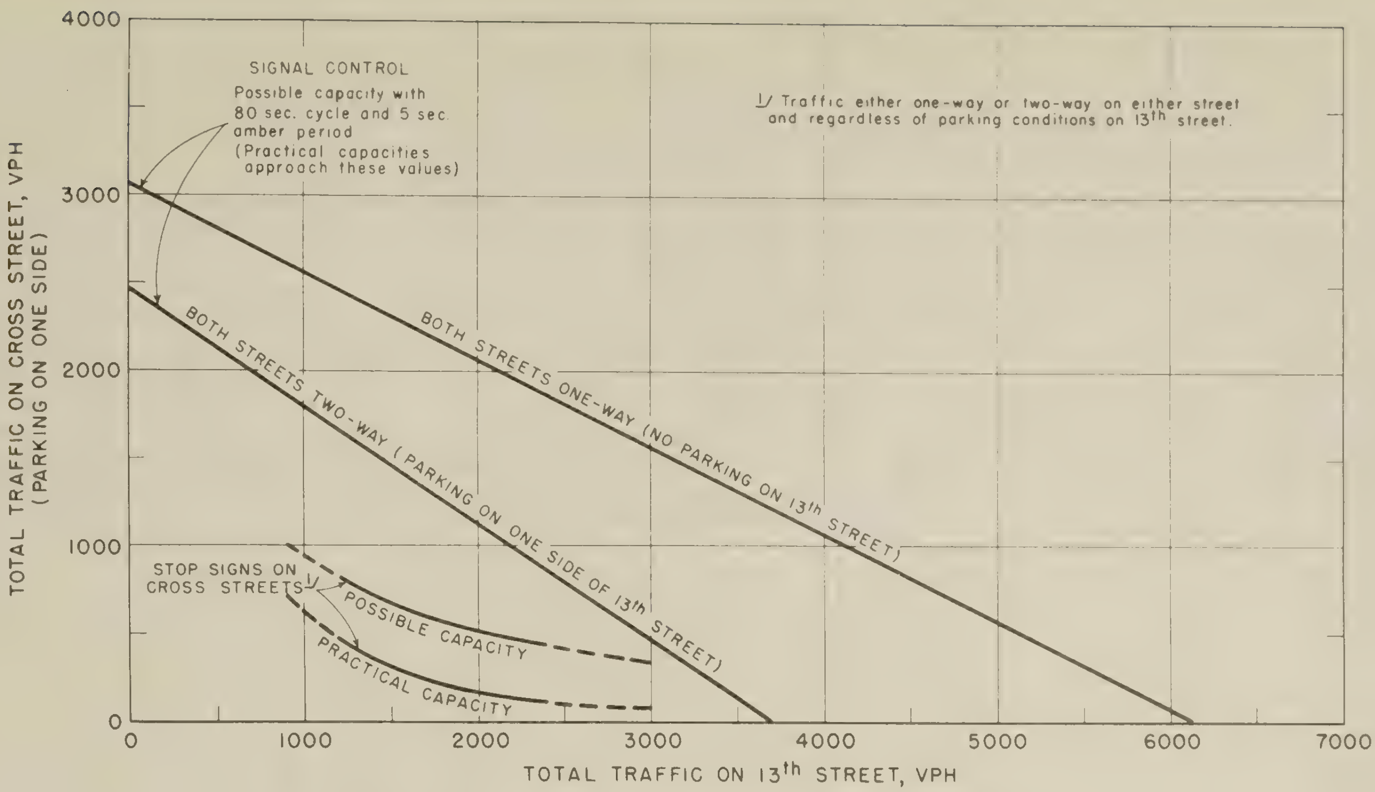

Figure 6.—Capacities for two types of intersection control.

At all traffic volumes there was practically no delay on the through street when stop signs were used to control traffic on the cross streets. Cross-street traffic experienced some delay even at the lower volumes, however, and this delay increased rapidly as the volume increased. With signal control both 13th Street traffic and cross-street traffic experienced some delay, yet this delay at all observed volumes was never greater for the cross streets than the delay with stop signs. The delay with signals was small as a result of the well-coordinated progressive system operating on the cross streets as well as on 13th Street.

While the difference in delay with stop signs and with signals was small at low volumes, it became very large at the higher volumes. For example, when the traffic volume on the through street approached 3,000 vehicles per hour, the delay to traffic on the cross street varied from 1½ to 6 minutes per vehicle with stop signs on the cross street, compared with 20 to 40 seconds per vehicle with signals. At a volume of 1,800 vehicles per hour on the through street and a cross-street volume of about 250 vehicles per hour, the average delay to all traffic on both streets was 10 seconds per vehicle with cross-street traffic controlled by stop signs and also when traffic was controlled by signals. At lower volumes the delays were only slightly less and were about equal for both types of control. At volumes above 1,800 vehicles per hour on the through street, the average delay to all traffic was greater with stop-sign control on the cross streets than with signal control.

Traffic on the through street was protected by stop signs on the cross streets, and had it not been impeded at other intersections along the route, the volume of traffic could theoretically have increased to 7,000 vehicles per hour. This type of control is not feasible, however, for such heavy traffic volumes on the through street because cross traffic is almost completely blocked.

When an intersection at which traffic on one of the streets is controlled by stop signs is adjacent to an intersection where traffic on the through street is controlled by signals, the capacity of the cross street is somewhat greater than it would be if there were no traffic signals in the immediate vicinity. The signal at the neighboring intersection creates gaps in the stream of traffic on the through street into which vehicles waiting at the stop sign may enter. At such locations traffic volumes greater than those indicated by the curves in figure 6 could probably be discharged from cross streets controlled by stop signs.

The traffic volumes observed in this study are evidently too great to be handled satisfactorily at intersections controlled by stop signs on the cross street. Even with moderately heavy volumes on the cross streets, this traffic was delayed as much or more by stop signs than by signals. At the higher volumes the delay with stop signs definitely became intolerable to cross-street traffic, although through traffic was completely unrestricted.

Stop-Sign Control on All Approaches

When stop signs were used on all approaches to the three intersections studied with this type of control, they were installed on the near side as well as the far side of the intersections, and advance warning signs were erected. On the one-way streets, signs were mounted on the left as well as the right-hand side of the street. Even under these conditions, it was necessary to discontinue the studies when the traffic volume reached the practical capacity of the intersections. The study had to be terminated after 13th Street had been operating one way for little more than an hour and before the height of the afternoon rush was reached. The frequency with which a police officer was required to regulate the various movements immediately before the study was terminated made it apparent that the practical capacities of the intersections for this type of control were reached.

Table 6 shows traffic volumes for the heaviest hour and for the heaviest 10-minute period of the study with stop signs on all intersection approaches. For each intersection, the volumes are shown separately for 13th Street and for the intersecting or cross street, and the combined total for the two is shown in the column, "Total for intersection." The average delay per vehicle during the period when the particular volume occurred is also shown in table 6. The peak volume on 13th Street did not always occur during the same time period as that for the cross street, and for this reason each intersection is listed twice in the table. The volumes for the cross streets as shown in the upper portion of the table are those observed during the period when 13th Street was carrying its peak load. In the lower portion of the table the volumes shown for 13th Street are those observed when the cross streets were at a peak.

The highest 10-minute volume was observed at the intersection of 13th Street and Columbia Road, with 13th Street operating as a one-way street northbound and Columbia Road, one-way westbound. The rate of flow on 13th Street during this period was 2,280 vehicles per hour, while the rate on Columbia Road during the same period was 762 vehicles per hour. The total volume for the intersection,

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Table 6.—Traffic volumes and delay per vehicle with stop-sign control on all approaches

Location	1-hour period						10-minute period					
	13th Street		Cross streets		Total for intersection		13th Street		Cross streets		Total for intersection	
	Traffic volume	Average delay	Traffic volume	Average delay	Traffic volume	Average delay	Traffic volume	Average delay	Traffic volume	Average delay	Traffic volume	Average delay
During Peak Volume On 13th Street [1]												
	V. p. h.	*Minutes*	*V. p. h.*	*Minutes*	*V. p. h.*	*Minutes*	*V. p. h.*	*Minutes*	*V. p. h.*	*Minutes*	*V. p. h.*	*Minutes*
13th Street, two way:												
At Park Road	1,231	0.44	512	0.54	1,743	0.47	1,476	0.31	642	0.39	2,118	0.33
At Columbia Road	1,400	.41	567	.27	1,967	.37	1,602	.49	480	.28	2,082	.44
At Harvard Street	1,389	.49	492	.47	1,881	.48	1,542	.47	594	.42	2,136	.46
13th Street, one way:												
At Park Road	1,502	.23	691	.80	2,193	.41	1,794	.25	810	1.05	2,604	.50
At Columbia Road	1,615	.31	668	.42	2,283	.34	2,280	.43	762	.52	3,042	.45
At Harvard Street	1,481	.38	653	.60	2,134	.45	1,890	.26	696	.86	2,586	.42
During Peak Volume on Two-Way Cross Street												
13th Street, two way:												
Park Road	1,147	0.35	565	0.54	1,712	0.41	1,476	0.31	642	0.39	2,118	0.33
13th Street, one way:												
Park Road	1,502	.23	691	.80	2,193	.41	1,794	.25	810	1.05	2,604	.50
During Peak Volumes on One-Way Cross Streets												
13th Street, two way:												
Columbia Road	1,400	0.41	567	0.27	1,967	0.37	1,314	0.36	684	0.37	1,998	0.36
Harvard Street	1,387	.53	495	.49	1,882	.52	1,530	.41	600	.36	2,130	.40
13th Street, one way:												
Columbia Road	1,615	.31	668	.42	2,283	.34	1,968	.45	822	.70	2,790	.52
Harvard Street	1,481	.38	653	.60	2,134	.45	1,818	.25	858	.45	2,676	.31

[1] Parking permitted on one side of 13th Street during two-way operation; parking prohibited during one-way operation.

3,042 vehicles per hour, is equivalent to an average of 507 vehicles per lane per hour for all lanes entering the intersection. The accompanying delay averaged 31 seconds per vehicle to traffic on the cross street and 26 seconds per vehicle to traffic on the through street. This is very close to the delay of 30 seconds which was used as the criterion for practical capacity with stop-sign control on the cross streets. It seems reasonable to assume that most drivers would consider a delay greater than this intolerable at an all-way stop intersection just as they do at a cross street with stop signs.

During the 10-minute periods of maximum traffic flow reported in table 6, it is noteworthy that the average volume per lane, including all approaches, was usually between 400 and 500 vehicles per hour. Most of the average delays to cross-street traffic observed during these same periods vary between 21 and 52 seconds. This indicates that the traffic load was between the practical and possible capacities of the intersections. It seems reasonable to assume that somewhat higher volumes might be carried with no increases in delay after a period of familiarization for the drivers. The practical capacity of these intersections with stop-sign control on all approaches and under the other existing conditions is therefore

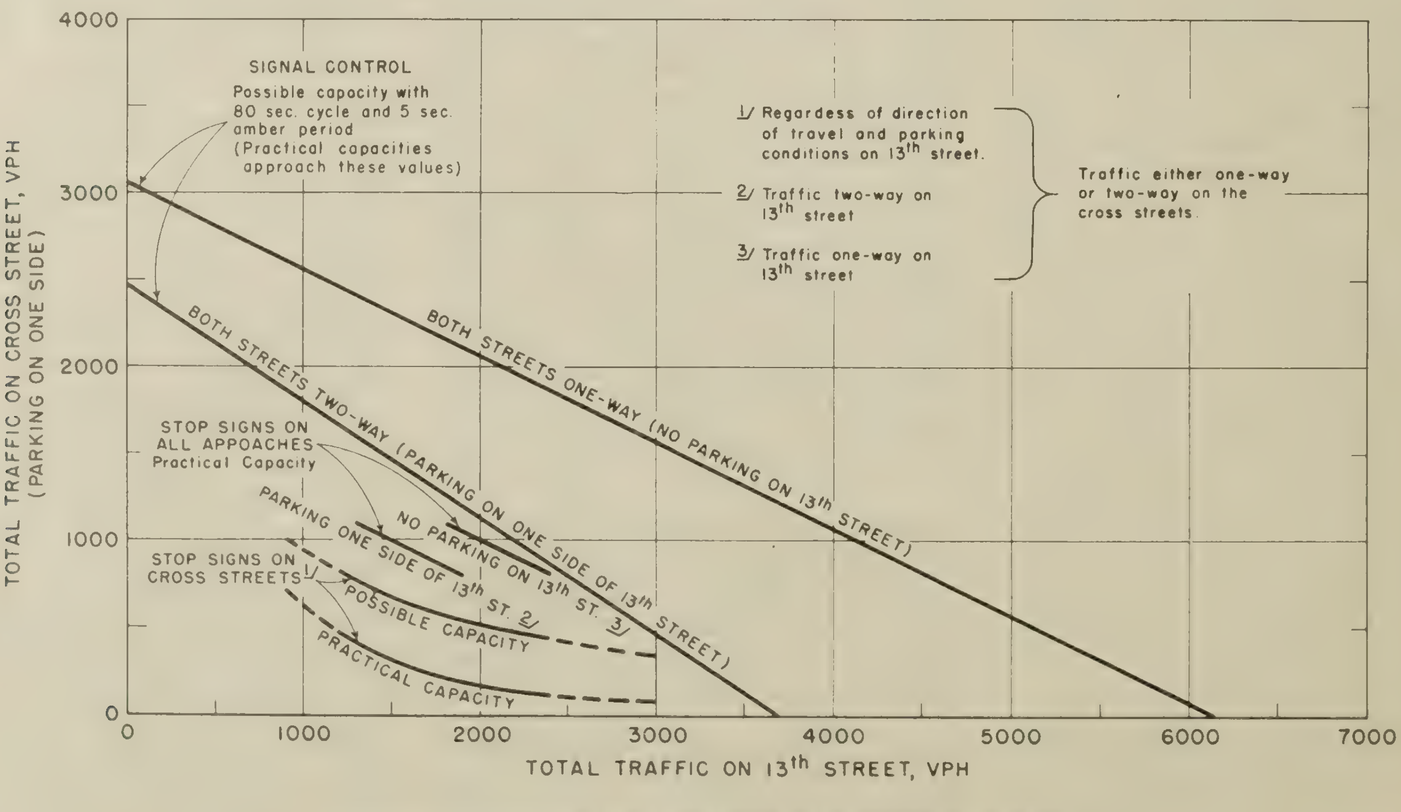

Figure 7.—Capacities for three types of intersection control.

somewhere near 500 vehicles per hour for each lane used by traffic entering the intersection. This capacity is based on a reasonable traffic delay but does not take into consideration pedestrian delays or accident hazards which would probably tend to reduce the 500 figure to a somewhat lower value. The few pedestrians at the intersections studied experienced long delays in crossing the streets even at moderate volumes.

Practical capacity for all-way stop control is compared with the capacities determined for the other types of control in figure 7. This shows that under the traffic conditions prevailing at these intersections the capacity with all-way stop control is greater than with cross-street stop-sign control and less than with signal control. As previously stated, the capacities with stop signs on the cross streets are limited to those that permit a reasonable movement of the cross-street traffic.

At intersections where both streets are two way, the practical capacity with four-way stop signs approaches the capacity with signals. The capacity with signals is much greater, however, than with stop signs on all approaches when both streets are one way.

The delay to traffic was compared for the three types of control. At the traffic volumes observed, the delay to all traffic was found to be more with stop signs on all approaches than with signal control. Also, when the volume on the through street was below 1,800 vehicles per hour, stop signs on all approaches resulted in a greater average delay to all traffic than did stop signs on the cross streets. When the volume on the through street was greater than 1,800 vehicles per hour, the average delay to all traffic was less with stop signs on all approaches than with stop signs on the cross streets.

An analysis of the delays shown in tables 5 and 6 indicates that the delay to cross-street traffic is far less when stop signs are used on all approaches than when used on the cross street only. An examination of other data obtained for lower traffic volumes shows this also to be the case for the lower volumes. The cross-street volumes in one direction during this study were never below 100 vehicles per hour. For cross-street volumes lower than this figure the relation between delay and type of stop-sign control might be quite different from that found in this study.

Use of the Kelly Ball as a Device for Measuring the Consistency of Concrete

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by WILLIAM E. GRIEB and ROBERT A. MARR, JR.
Highway Physical Research Engineers

The Kelly ball test is a simple field method for determining the consistency of plastic concrete. It is made by measuring the penetration of a 30-pound metal "ball" into the surface of the concrete. This test can be made on the concrete in place which results in a considerable savings in time and effort compared with the slump test. The data reported in this article show good correlation between the slump and Kelly ball penetration tests.

THE American Society for Testing Materials standard slump test [1] has been used for many years as a measure of the consistency of fresh concrete in the laboratory and on the job. As a laboratory procedure it is reasonably satisfactory. In the field, especially on paving work, it has several disadvantages and the most serious of these is the time required to make tbe test. Others are the necessity for careful selection of samples and the close attention to details of the technique required to obtain reasonably accurate results.

Realizing the disadvantages of the slump test as a field control method, Prof. J. W. Kelly of the Department of Civil Engineering, University of California, developed a penetration device that is rugged and portable. This test was recently made ASTM Tentative Standard C 360–55T.

Professors Kelly and Polivka of the University of California gave the following account of the development of this test in an article published recently.[2]

> The test was developed in the Engineering Materials Laboratory of the University of California at Berkeley as an outgrowth of an attempt to devise a simple test for workability of

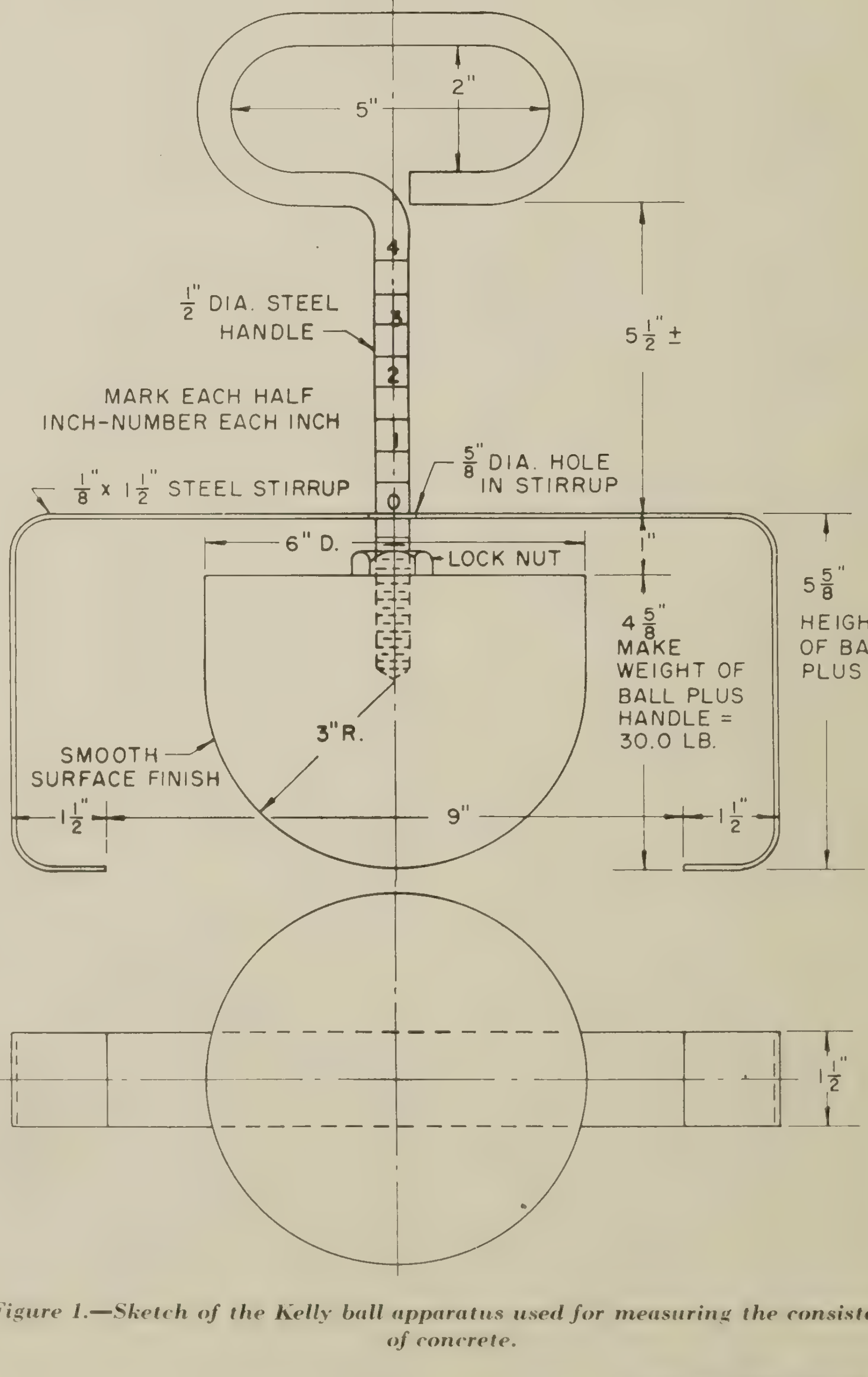

Figure 1.—Sketch of the Kelly ball apparatus used for measuring the consistency of concrete.

[1] *Standard method of test for slump of portland-cement concrete,* ASTM Designation: C 143–52. ASTM Standards, 1952, Part 3, pp. 1086–1087.

[2] *Ball test for field control of concrete consistency,* by J. W. Kelly and Milos Polivka. Journal of the American Concrete Institute, May 1955, vol. 26, No. 9, pp. 881–888.

concrete. Workability is an elusive property, and early trials with various balls showed little correlation with the more elaborate tests in laboratory use. However, it was observed that static ball penetration correlated rather well with slump and it became apparent that the penetration test measured some property similar to slump—a property which had been termed "consistency" but which is now called merely "slump" in ASTM C 143. It is the significant property which is measurable in the field for practical purposes.

A 6-inch ball was considered to be the smallest that would integrate the resistance to penetration over several pieces of aggregate, and a 30-pound weight was found to be the lightest that would penetrate reproducibly the stiffest mixes of plastic concrete. This combination of area (or displaced volume) and force has been found applicable even to harsh concrete containing 2½-inch aggregate and having a nominal slump, when wet-screened, of 1½ inches. The apparatus has also been used on mass concrete containing 0 to 6-inch crushed aggregate by making the penetration test only on areas which had been found by probing to be free from the larger pieces of aggregate. A 20-pound weight on a 6-inch ball has sometimes been used for lightweight concrete.

Penetration tests have been developed independently in other countries. A static test used in Spain employs a weight on a spherical tip and having a flared edge so that the plunger will not sink too deep into wet concrete. The German Committee on Reinforced Concrete has adopted an impact test suitable for stiff mixes or mixes of low cement content; it consists in dropping a 33-pound plunger having a 4-inch hemispherical tip 8 inches onto the surface. In England, the Wigmore consistometer employs a metal ball set on the surface of a concrete sample which is vibrated on a table.

* * * * *

The static ball test was introduced to field use by E. L. Howard, testing engineer, Pacific Coast Aggregates, Inc., San Francisco. His experience with the variety of mixes used in ready-mixed concrete was so successful that it encouraged the authors to report the test to ASTM Committee C-9 at its San Francisco meeting in 1949. Mr. Howard has continued to contribute to its field development, and is convinced that it will eventually replace the slump test.

Many other organizations have adopted the ball test, and hundreds of balls are in use throughout the country. The California Division of Highways has adopted it as a standard for field use on pavement construction. At least two other State highway departments—North Carolina and Colorado—are using it extensively. The Waterways Experiment Station, Concrete Division, U. S. Army Corps of Engineers, has adopted it as an alternative standard.

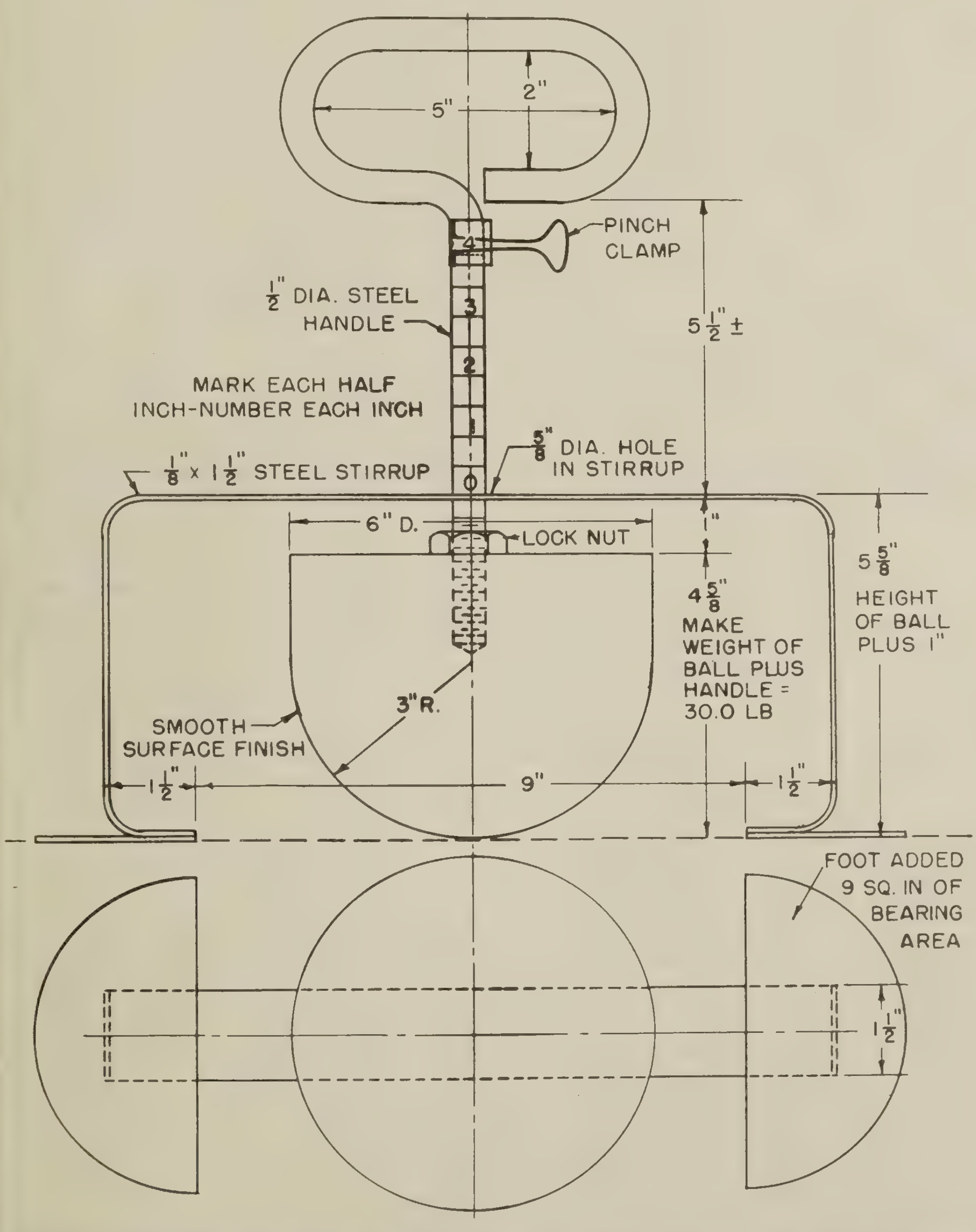

Figure 2.—Sketch of the Kelly ball apparatus after modification by the Bureau of Public Roads.

Kelly Ball Apparatus

The apparatus is popularly known as the "Kelly ball." It is made by machining into a hemisphere, one end of a solid right cylinder 6 inches in diameter and 4⅝ inches in height. It is fitted with a graduated vertical rod ½ inch in diameter which serves as a measuring scale and a handle. The vertical portion of the rod is graduated in ½-inch units with each inch numbered. The ball is guided by a stirrup or frame which also serves as a reference line in measuring penetration of the ball into the plastic concrete. The zero on the graduated handle coincides with the top of the frame when the apparatus rests on a level rigid surface. The weight of the ball and handle is 30 pounds. A sketch of the original apparatus is shown in figure 1.

Modification of the Kelly Ball

The Kelly ball equipment used by the Bureau of Public Roads differs from the original in the following details:

To prevent the reference frame from tilting, the bearing area of each foot of the frame (originally specified as 1½ square inches) was increased by the addition of semicircular bearing plates of 5-inch diameter (area approximately 9¾ square inches). The clear distance between the feet was maintained at 9 inches as originally specified (see fig. 2). This

Figure 3.—Kelly ball penetration test being performed under field conditions.

change is included in the ASTM Tentative Method C 360–55T.

To facilitate reading the depth of penetration of the ball, a vertically movable pinch clamp was fastened to the graduated handle. This is clamped at the top of the handle before making the test and is lowered to make contact with the top of the frame after the ball has penetrated the surface of the concrete. The apparatus is then removed from the concrete and the penetration reading is made by reading the position of the clamp on the rod. This makes the test procedure more convenient for the operator and avoids any possible delay in the concreting operation.

Use of the Kelly Ball

Plastic concrete can be tested with the Kelly ball after placement in the forms and prior to any manipulation, or in suitable containers such as tubs, pans, wheelbarrows, or buggies.

In making the test, the surface of the concrete is smoothed and leveled quickly by the use of a small wooden float or screed. The surface is worked as little as possible to avoid formation of a mortar layer. During the test, the adjacent concrete should not be vibrated, jarred, or agitated.

The ball is held vertically by the handle in very light contact with the leveled concrete surface and with the zero on the rod coincident with the top of the frame. The handle is then released and the depth of penetration of the ball into the concrete is estimated on the graduated rod to the nearest 0.1 inch (see fig. 3). A minimum of three readings should be taken from a batch or location. No correction is made for any slight settlement of the feet of the frame. The test requires less than ½ minute to perform, which allows the operator sufficient time to work where the concrete is being discharged from the mixer without delaying the placing and finishing operations.

Experience has indicated that the minimum depth of concrete tested should be 6 inches for a maximum size of coarse aggregate of 2 inches or less. When larger coarse aggregate is used, the minimum depth should be three times the nominal maximum size of coarse aggregate. When testing concrete placed inside forms as in piers, walls, etc., a minimum horizontal distance of 9 inches from the face of the form to the point tested should be maintained to avoid boundary effects. For concrete discharged on the subgrade in pavement work, no test should be made nearer than 9 inches to the form edge of the leveled surface of the concrete. For a second test in the same batch of concrete, the foot of the stirrup should be at least 6 inches from the point where the foot rested in the first test.

Field Tests

A limited number of tests were made on several paving projects to correlate Kelly ball penetration readings with the corresponding slumps. The concrete used on the first of these projects contained 6½ sacks of cement per cubic yard, 2-inch maximum size gravel, and about 5 percent air. Batches were selected so that tests could be made without delaying the progress of the work, and so that some adjustments could be made in the water content and consistency of the batches. The concrete was discharged on the subgrade in a pile 8 to 10 inches in height. A sample for the slump test was taken and thoroughly remixed in a pan.

The Kelly ball penetration test was made on the concrete in place on the subgrade prior to any manipulation. The top surface of the concrete was leveled with a wood float, the apparatus set on that surface, and the penetration read. The water content of the concrete was varied in order to obtain a greater range in consistency. Usually two or three Kelly ball penetration readings were made for comparison with the reading of each slump. Test data taken over a 2-day period are tabulated in table 1 and are shown graphically in figure 4 (A). The average penetration was 2.4 inches, the average slump was 3.6 inches, and the ratio of penetration to slump was 1 to 1.5.

The concrete on the second project had the same cement content and the same maximum size coarse aggregate as that on the first project. The aggregates were from a different source and the concrete was mixed in a ready-mix truck. The test data taken over a 7-day period are tabulated in table 1 and are shown graphically in figure 4 (B). For this job the average penetration was 2.3 inches and the average slump was 3.2 inches. The ratio of penetration to slump was 1 to 1.4.

The concrete in the third project was similar to that in the second project, but the maximum size of coarse aggregate was 1 inch instead of 2 inches. The test data taken over a 6-day period are tabulated in table 1 and illustrated in figure 4 (C). The average penetration and slump for this job were 2.3 and 3.0 inches, respectively. The penetration slump ratio was 1 to 1.3.

The data in table 1 and figure 4 (field tests) show a reasonable correlation between Kelly ball penetration and slump readings for a range in slump of 1 to 5½ inches. The average Kelly ball reading multiplied by 1.4 would provide a fair estimate of the corresponding slump for the range indicated and materials used. These limited data indicate that for a maximum size of coarse aggregate of 1 inch this ratio might be reduced to 1.3.

Laboratory Tests

Slump tests and Kelly ball penetration readings were also made on concrete mixed in the laboratory and placed in steel forms for the fabrication of slabs for structural tests containing approximately 33 cubic

Table 1.—Correlation of Kelly ball penetration test and slump test for consistency of concrete under field conditions [1]

Project No. 1			Project No. 2			Project No. 3		
Kelly ball penetration		Slump test	Kelly ball penetration		Slump test	Kelly ball penetration		Slump test
Individual tests	Average, all tests		Individual tests	Average, all tests		Individual tests	Average, all tests	
Inches	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*
1.1, 1.2, 1.2	1.2	2.3	1.6	1.6	2.7	1.6	1.6	1.8
1.4, 1.4, 1.1	1.3	.7	1.6, 1.6	1.6	2.0	1.6, 1.5	1.6	1.8
1.8, 1.0	1.4	1.7	1.6, 1.7	1.6	2.6	1.8, 1.6	1.7	2.7
1.4, 1.6	1.5	2.6	1.6, 2.1	1.8	2.4	1.8, 1.9, 1.8	1.8	2.2
1.7, 1.9, 1.5	1.7	2.3	2.0, 1.6	1.8	2.2	2.0, 1.7	1.8	2.7
2.0	2.0	3.3	2.3, 1.3	1.8	2.5	1.9, 2.4, 1.8	2.0	2.7
2.2, 2.2, 2.4	2.3	3.7	2.0, 1.8	1.9	2.8	2.0, 1.9, 2.2	2.0	3.4
2.5, 2.6, 1.9	2.3	3.9	1.7, 2.2	2.0	3.1	2.1, 2.2, 2.0	2.1	2.8
2.3, 2.5	2.4	3.7	2.1	2.1	4.0	2.1, 2.4	2.2	2.8
2.5, 2.4	2.4	4.7	2.2, 2.0	2.1	3.0	2.3, 2.2	2.2	2.5
2.6, 2.8	2.7	3.7	2.2, 2.2	2.2	2.8	2.4, 2.2	2.3	2.6
2.7	2.7	4.3	2.2, 2.2	2.2	3.0	2.3, 2.5	2.4	2.8
2.8	2.8	4.0	2.1, 2.8	2.4	2.8	2.8, 2.8	2.8	3.3
2.7, 3.5, 2.6	2.9	4.7	2.6, 2.1	2.4	3.6	3.0, 2.6	2.8	3.8
3.2, 3.4	3.3	4.5	2.4, 2.6	2.5	3.4	2.8, 2.9, 3.1	2.9	4.3
3.7, 3.1	3.4	4.1	2.8, 2.5	2.6	3.4	3.4, 3.0	3.2	3.8
3.7	3.7	4.4	3.1, 2.0	2.6	3.0	3.8, 4.0	3.9	4.6
4.2, 4.7, 3.7	4.2	5.3	2.5, 2.9	2.7	3.6			
			2.8, 2.9	2.8	5.5			
			3.5, 3.3	3.4	4.8			
			3.6, 3.4	3.5	4.5			
Average [2]	2.4	3.6		2.3	3.2		2.3	3.0

[1] Mixture used for project Nos. 1–2: 2-inch maximum size gravel, 6½ sacks of cement per cubic yard, and 5 percent air content. Same cement content and air content used for project No. 3; maximum size gravel was 1 inch.

[2] Ratios of penetration to slump: project No. 1, 1:1.5; project No. 2, 1:1.4; and project No. 3, 1:1.3.

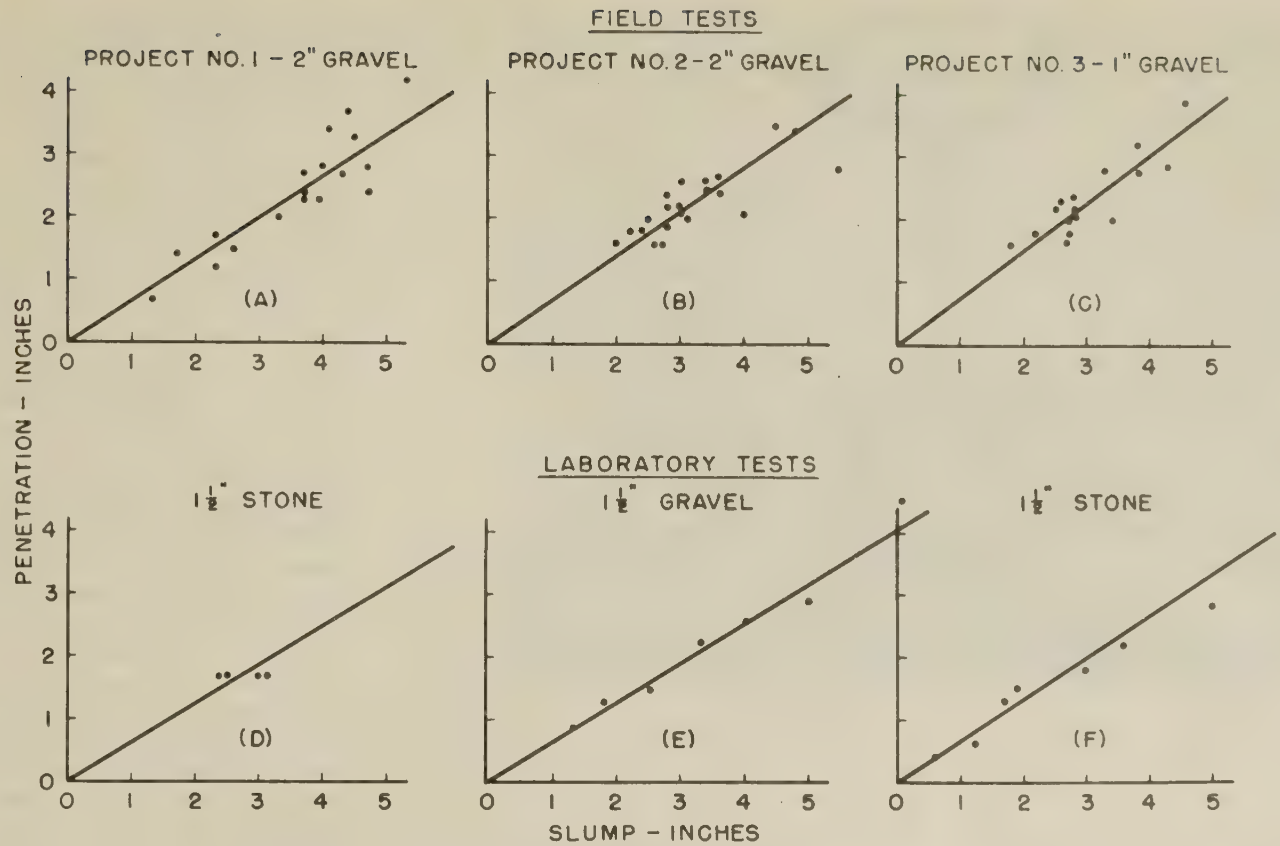

Figure 4.—Correlation of Kelly ball penetration and slump tests.

feet of concrete. The water content was the same for all batches. The concrete was non-air-entrained and contained 6 sacks of cement per cubic yard with crushed-stone coarse aggregate of 1½-inch maximum size. The results of these tests are shown in table 2 and in figure 4 (D). This figure does not have much significance due to the limited number of tests. For these tests the average slump was 1.6 times the Kelly ball reading.

Laboratory tests were made on concrete mixes using both gravel and crushed-stone coarse aggregates of 1½-inch maximum size and having variable slumps. The concrete was non-air-entrained and contained 6 sacks of cement per cubic yard. The water content was varied to produce slumps ranging from 1 to 6 inches. The results of these tests are tabulated in table 2 and are shown graphically in figure 4 (E) for gravel and figure 4 (F) for crushed stone. The ratios of the average Kelly ball penetration to slump were 1 to 1.6 for crushed stone and 1 to 1.5 for gravel. These tests show approximately the same relation between the slump and Kelly ball readings as were shown for the field tests where similar materials were used.

The results of field and laboratory tests discussed in this article are in reasonably good agreement with those obtained by other investigators. In an extensive series of tests conducted by the Concrete Division, Waterways Experiment Station of the U. S. Army Corps of Engineers at Jackson, Miss., the average ratio of slump to penetration of 1.8 was reported as compared with 1.5 and 1.6 obtained in this study. Walker and Bloem [3] in an unpublished report gave the average ratio of slump to penetration of 1.66 for over 250 tests.

Table 2.—Correlation of Kelly ball penetration test and slump test for consistency of concrete under laboratory conditions

Identification	Fixed water content [1]		Varying water content [2]			
			Mix No. 1, stone		Mix No. 2, gravel	
	Kelly ball penetration	Slump	Kelly ball penetration	Slump	Kelly ball penetration	Slump
	Inches	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*
A	1.4	----------	0.4	0.6	0.9	1.3
B	1.5	----------	.6	1.2	1.3	1.8
C	1.5	----------	1.3	1.7	1.5	2.5
D	1.7	2.4	1.5	1.9	2.3	3.3
E	1.7	2.5	1.8	3.0	2.6	4.0
F	1.7	3.0	2.2	3.6	2.9	5.0
G	1.7	3.1	2.8	5.0	3.8	5.6
H	1.7	----------	----------	----------	4.5	6.5
I	1.7	----------	----------	----------	----------	----------
J	2.0	----------	----------	----------	----------	----------
K	2.2	----------	----------	----------	----------	----------
Average [3]	1.7	2.8	1.5	2.4	2.5	3.8

[1] Mixture used: 1½-inch maximum size crushed stone, and 6 sacks of non-air-entrained concrete per cubic yard. Each value represents one test.

[2] Mixture used: 1½-inch maximum size coarse aggregate, and 6 sacks of non-air-entrained concrete per cubic yard. Each value for Kelly ball penetration is an average of 6 tests and each value for slump is an average of 2 tests.

[3] Ratios of penetration to slump: for fixed water content, 1:1.6; for varying water content, 1:1.6 for stone and 1:1.5 for gravel.

Advantages of the Kelly Ball Test

On any particular project using specific materials, a limited number of tests will correlate the Kelly ball readings with the corresponding slump tests sufficiently to permit using the Kelly ball for the control of the consistency of the concrete when a slump range has been specified.

The following comments are made on the Kelly ball test as a replacement for the slump test for measuring the consistency and uniformity of concrete in the field:

1. The concrete may be tested in place, therefore the selection or preparation of a sample is eliminated.

2. Three or more Kelly ball tests can be made at a selected location in less time and with less effort than is required for one slump test. Due to the speed with which the test can be made, the operator can work where the concrete is being discharged from the mixer without delaying paving or finishing operations.

3. Making the consistency test easier and faster should encourage more frequent testing and should be helpful in controlling the uniformity of the concrete.

[3] Stanton Walker and D. L. Bloem, Director of Engineering and Assistant Director of Engineering, respectively, National Sand and Gravel Association and National Ready Mixed Concrete Association.

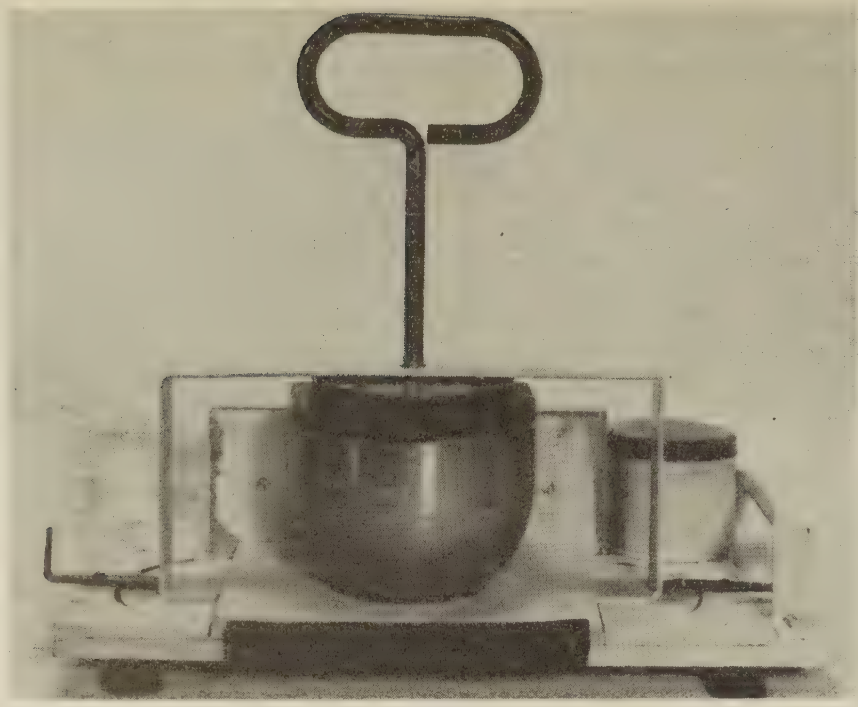

Figure 5.—Field carrying kit for Kelly ball apparatus.

4. The apparatus can be maintained in usable condition between tests by merely wiping with an oily rag.

5. The slump test is not practical for use in testing concrete with a maximum size of coarse aggregate over 2 inches. The Kelly ball penetration test may be used on concrete containing larger aggregate if a sufficient volume is available to provide adequate depth.

Field Kit

For ease in transporting, the ball with wood float and a base plate can be readily assembled into a compact field kit as shown in figure 5.

The wooden float is used to level the concrete at the area to be tested. A tin "rag can" provides a place to keep oily cloth or waste to wipe the ball clean after each test. The ball should not be placed too near the side forms or the edge of a pile of concrete. In order to meet this requirement, the operator frequently must place one foot into the wet concrete. The carrying base plate is designed as a footboard to support the weight of the operator on the plastic concrete.

The apparatus may be built in any machine shop. However, it has been adopted as a tentative standard by the American Society for Testing Materials and may be offered for sale by the leading instrument companies.

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An Improved Sulfate Soundness Test for Aggregates

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

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The present sulfate soundness test method, AASHO Method T 104–46 or ASTM Method C 88–46T, permits in tests of coarse aggregate the determination of six different values any one of which may be considered as "standard." Different laboratories may not agree on the method to be used in testing a given sample of aggregate. An unwarranted dependence is placed on the results of the test. To correct this practice, it is recommended that the method be used for the acceptance but not the rejection of aggregates, that the sample tested be restricted to one size of coarse or fine aggregate, that the drying oven meet requirements for performance, that the loss for coarse aggregate be determined with sieves having openings one-half the size of those in the original retaining sieves, and that certain minor changes be made. It is believed that these changes will result in better agreement between laboratories conducting check tests, and that the test result will permit a reliable appraisal of the soundness of the material under test.

ONE of the first descriptions in American roadbuilding literature of the sodium sulfate soundness test for aggregate appeared in Bulletin 1216 of the U. S. Department of Agriculture. In this bulletin, the tentative standard methods of sampling and testing highway materials were described as adopted in 1922 by the American Association of State Highway Officials. The description of the soundness test is remarkable in its brevity and clearness. It reads as follows:

> Immerse 10 small pieces (total weight about 1,000 grams) of the rock in a saturated solution at 70° F. of sodium sulfate (Na_2SO_4) for 20 hours, after which place them for 4 hours in a drying oven maintained at 100° C. Repeat the treatment five times. Note the condition of the rock as to soundness at the end of the test. Samples which exhibit marked checking, cracking, or disintegration shall be considered to have failed in this test.

With this publication, a long program of study of the sodium sulfate soundness test began. Change after change was made in the method with a view toward broadening it to cover both fine and coarse aggregate, and to standardize it in all its details. In addition, some users saw in the method a chance to develop an acceptance test of aggregate which could be made in a relatively short time in comparison with that required for a freezing and thawing test. With little regard to the actual processes involved in the crystallization of sulfate salts, or of the proper interpretation of the test results, the use of the test for the acceptance or rejection of aggregates was solemnly indorsed. Under pressure for a quick decision, the highway testing engineer turned to the sulfate soundness test as a measure of the acceptability of aggregates. With further misunderstanding of the applicability of the method, requirements for the loss in the sulfate soundness test were written in specifications for aggregates. The use of the method and of requirements based thereon was widely adopted by construction authorities.

Laboratories found difficulty in obtaining reproducible test results, or in testing materials in strict accordance with the requirements of the method. An attempt was made to adjust the method to permit the testing of aggregates of various gradations.

Recommendations

With a view toward securing a dependable, reproducible test method which can be used for a rapid appraisal of the soundness of aggregates, it is recommended that the following changes be made in AASHO Method T 104–46 or the similar ASTM Method C 88–46T:

1. Revise the scope to indicate that the method is indicatory only; that it may be used for acceptance of material but that rejection should be based on other determinations such as freezing and thawing tests, inspection of ledges at the quarry (if the material is rock), or inspection of concrete prepared with the material.
2. Revise the requirements for the oven by the addition of performance determinations.
3. Revise the requirements for the samples to include only the ½-inch to ⅜-inch size of coarse aggregate, and the No. 8 to No. 16 size of fine aggregate.
4. Determine the loss for coarse aggregates with the half-size (No. 4) sieve, but continue the use of the original retaining sieve for fine aggregates.
5. Revise the requirements for the maintenance of the solution to emphasize the necessity for thorough stirring and the breaking of caked salt.
6. Revise the requirements for sample containers to permit the use of perforated or nonperforated containers.

To accompany these changes in the method of test, a change from 12 to 7 percent should be made in the requirements for the loss in the sodium sulfate soundness test given in AASHO Specification M 80 or ASTM Specification C 33 for coarse aggregate for concrete. A similar change for fine aggregate in ASTM Specification C 33 or AASHO Specification M 6 should be made, increasing the allowable loss from 10 to 13.5 percent.

Discussion of Earlier Tests

The first mention of the testing of graded coarse aggregate appeared in the 1931 Proceedings of the American Society for Testing Materials as Method C 89–31T.[1] This method required that the aggregate be separated and tested in the following sizes:

Size	*Minimum weight of sample, grams*
No. 4 (0.185 in.) to ⅜ in.	100
⅜ in. to ¾ in.	300
¾ in. to 1½ in.	1, 500
1½ in. to 2½ in.	3, 000
Larger sizes by 1-inch spread	3, 000

The reason for the selection of these sizes is not known, but it appears that the use of the so-called fineness modulus sieves was considered desirable. However, the largest size does not include the 3-inch sieve as used in the fineness modulus series of sieves. It is possible that a difference of one inch between the sizes of the two sieves used to prepare the test sample was as great as was deemed advisable.

Difficulties in obtaining check results of soundness tests between laboratories or even in the same laboratory were the general rule. Possibly in an attempt to correct this, the grading of the sample was more closely defined in AASHO Method T 104–42 and ASTM Method C 88–41T. In these revised methods,

[1] Methods C 88 for fine aggregate and C 89 for coarse aggregate were combined in 1937 and identified as C 88. AASHO methods T 75 and T 76 for the sulfate soundness test of fine and coarse aggregates, respectively, were combined in 1938 and identified as T 104.

the same sizes of coarse aggregate as shown in the 1931 ASTM method were used, but each of the three sizes between the ⅜- and 2½-inch sieves was separated into a coarse and a fine fraction, and these fractions recombined in a prescribed ratio. Some of the minimum weights for sizes tested were also increased. The requirements for the composition of the sample tested are shown in table 1.

The sizings established in this revision did not prove satisfactory. Many aggregates for concrete did not contain material larger than 2 inches, but did contain enough of the 1½- to 2-inch size to require that this size be tested. Considerable confusion developed regarding the sample to be prepared for test. Some laboratories ignored the requirement that the sample contain some 2- to 2½-inch material. Since it was not present in the aggregate as submitted, it could not be tested. These laboratories then tested the 1½- to 2-inch size. Other laboratories adhered more closely to the requirements of the method, and did not test the 1½- to 2-inch size if there was no aggregate larger than 2 inches. The results of check tests between laboratories were not in good agreement and the method was repeatedly condemned as unsatisfactory.

Use of Alternate Gradings

To alleviate some of the troubles found, a second grading for the preparation of the sample was added to the method in 1946 by both the AASHO and the ASTM. This grading, described as Alternate A, required the combination of sizes shown in table 1.

At the same time, a third grading was adopted, possibly to satisfy criticism of the use of the two other combinations of sizes. This grading was identified as Alternate B, and required use of the sizes and weights of sample shown in table 1.

All three of the gradings for the test sample are now equally standardized with respect to this method of test, and confusion may develop if one is designated here as the standard. As two of the gradings are now described as Alternates A and B, the other which was the first grading adopted is identified here as Grading 1.

Determination of Loss

Provision for the determination of the loss by either of two methods can be found in the 1941 and subsequent revisions of the sulfate soundness test. The loss may be measured as the material passing the sieve on which the sample was originally retained, or the material passing a sieve having openings one-half of that size. The initial retaining sieve considered here is that for the combination of sizes of fragment which are tested together. In Grading 1, for example, the original retaining sieve for the ¾- to 1-inch and the 1- to 1½-inch sizes would be the ¾-inch sieve, and the half-size sieve would be the ⅜-inch sieve.

The use of the half-size sieve to determine the loss in the test has three excellent features. It should permit a nominal variation between sieves used in different laboratories to prepare the test sample, and will not require an excessively meticulous separation by size and minute attention to particles of flat or elongated shape. Secondly, it should prevent the accidental inclusion in the loss of pieces which are structurally sound, but due to handling during the test, may suffer minor flaking or chipping of corners and edges. At the completion of the test, these pieces may pass the initial retaining sieve and be included, quite incorrectly, in the loss. Finally, the material passing this half-size sieve would be material which had suffered multiple cracking or disintegration. It would be material which actually was unsound, instead of material which had suffered minor surface damage or had broken along incipient fractures developed in the crushing and handling of the aggregate at the plant.

Table 1.—Requirements for sample of aggregate prepared for sulfate soundness test

Size of fragment	Grading 1	Alternate A	Alternate B
Inches	*Grams*	*Grams*	*Grams*
No. 4–⅜	300	300	300
⅜–½	[1] 1,000		500
½–¾		[1] 1,500	750
¾–1	[1] 1,500		1,000
1–1½		[2] 3,000	1,500
1½–2	[2] 3,000		2,000
2–2½		----	----
Larger than specified by 1-inch spread	3,000	3,000	3,000

[1] 33 percent of smaller size and 67 percent of larger size.
[2] 50 percent of each size.

Variation in Loss Due to Testing Procedure

The method of test for coarse aggregate as now written permits the use of three different gradings, and the determination of the loss by either of two methods. If a sample of certain but not unusual size is tested, there may be some doubt, as previously mentioned, in the sizes to be tested. An aggregate of 2-inch to No. 4 size, containing 5 percent or more of the 2- to 1½-inch size, would be included in this category. If it is desired to obtain the maximum information of the quality of the sample, many laboratories may prepare the test material in accordance with Grading 1, using presumably a 3,000-gram sample of 2- to 1½-inch aggregate. A summary of the test samples which might be prepared with this material is given in table 2.

It is quite unusual for different sizes of a sample of aggregate to have the same or nearly the same loss in the sulfate soundness test. Consequently samples of one aggregate prepared with the three different gradings may have markedly different losses. The loss as determined with the half-size sieve is only in rare cases of the same order of magnitude as that found with the original retaining sieve. Acceptance or rejection of an aggregate may then be governed by the grading used in the test, and the method of determining the loss.

An illustration of this is presented in tables 3–6. The values given in table 3 were obtained in a routine test of a sample of gravel. The grading of the sample as received conformed to AASHO Specification M 80–49 for coarse aggregate. This grading is such that the sample tested may be prepared to have either of the combinations of sizes described here as Grading 1 and Alternate A, or the sample may be tested by the individual size method identified as Alternate B.

Alternate B grading

The sample was tested by the individual size method (Alternate B) to obtain the most information regarding its quality. At the completion of the fifth immersion period, the sample was dried, washed free of the sulfate salt, again dried, and each separate size sieved to determine the losses given in table 3. An average loss for the sample, weighted with respect to the grading of the material as received, was then computed from the losses passing the original retaining (full-size) sieves, and from the losses passing the half-size sieves. These values are given in table 4. Average losses were also computed as if the sample had been tested using Grading 1, or using the Alternate A grading.

Grading 1

The computation of the average weighted loss for Grading 1 is shown in detail in table 5.

Table 2.—Requirements for sample of 2-inch to No. 4 aggregate prepared for sulfate soundness test (AASHO Method T 104–46)

Size of fragment	Grading 1: Minimum weight	Grading 1: Sieve used for loss [1]	Alternate A: Minimum weight	Alternate A: Sieve used for loss [1]	Alternate B: Minimum weight	Alternate B: Sieve used for loss [1]
Inches	*Grams*	*Inches*	*Grams*	*Inches*	*Grams*	*Inches*
2–1½	3,000	1½	[2] 3,000	1	2,000	1½
1½–1	[3] 1,500	¾			1,500	1
1–¾			[3] 1,500	½	1,000	¾
¾–½	[3] 1,000	⅜			750	½
½–⅜			[4] 300	No. 4	500	⅜
⅜–No. 4	300	No. 4			300	No. 4
Total minimum weight	5,800	--------	4,800	--------	6,050	--------

[1] A sieve having an opening one-half of the size given may be used as an alternate.
[2] 50 percent of each size.
[3] 67 percent of larger size and 33 percent of smaller size.
[4] Applies to ½-inch to No. 4 aggregate; no requirements given for sizes separated by the ⅜-inch sieve.

Table 3.—Grading of sample and loss in sulfate soundness test

Size tested	Grading as received	Loss: Amount passing sieve after test as a percentage of fraction originally retained on each sieve							
		1½ in.	1 in.	¾ in.	½ in.	⅜ in.	No. 3	No. 4	No. 8
Inches	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
2–1½	5	19.4	10.6	4.8	2.3	------	------	------	------
1½–1	21	------	11.8	11.2	10.9	9.5	------	------	------
1–¾	19	------	------	17.2	12.0	8.1	5.0	------	------
¾–½	17	------	------	------	18.1	13.1	8.9	5.4	------
½–⅜	16	------	------	------	------	17.0	11.8	8.8	6.8
⅜–No. 4	22	------	------	------	------	------	------	11.3	7.0

This grading requires that the 1½- to 1-inch and 1- to ¾-inch sizes be tested together as well as the ¾- to ½-inch and ½- to ⅜-inch sizes. In each case, the tested material contains 67 percent of the larger size and 33 percent of the smaller size. The original retaining sieve for the combination of the 1½- to 1-inch and 1- to ¾-inch sizes is considered here to be the ¾-inch sieve, and a half-size sieve would be the ⅜-inch sieve. Similarily, the ⅜-inch and No. 4 sieves are considered to be the original retaining and half-size sieves, respectively, for the combination of the ¾- to ½-inch and ½- to ⅜-inch sizes.

The effect of using these combined sizes can be obtained by the use of conversion factors as shown in table 5. The actual loss for the 1½- to 1-inch size passing the ¾-inch sieve is multiplied by 0.67 and added to 0.33 times the actual loss for the 1- to ¾-inch size passing the same sieve. The actual losses passing the ⅜-inch sieve for the ¾- to ½-inch, and ½- to ⅜-inch sizes are similarly combined. The losses for these combined sizes as well as those for the two sizes which were not combined, are used to compute the weighted average loss.

Alternate A grading

Table 6 shows the computation of the weighted average for the sample as if it had been tested using the Alternate A grading. The 2- to 1½-inch and 1½- to 1-inch sizes are combined in equal weights for testing, but the 1- to ¾-inch and ¾- to ½-inch sizes are combined in the 67–33 ratio. In the table, appropriate conversion factors are used. The Alternate A grading does not show any separation of the ½-inch to No. 4 size. As this size was separated on the ⅜-inch sieve and so tested, the losses for the ½- to ⅜-inch and ⅜-inch to No. 4 sizes have been combined in the same ratio that these sizes occurred in the original sample.

The results given in tables 4–6 are summarized in table 7. If the sample is tested by the Grading 1 method and the loss considered as the material passing the initial retaining sieve, the material will not meet the the requirement of 12 percent given in ASTM Specification C 33–52T or in AASHO Specification M 80–51 for coarse aggregate for portland cement concrete. If the sample is tested with the grading given in the specification as Alternate A, the loss in the test will meet the requirements of the specification. Should the sample be tested using the individual sizes permitted under Alternate B, it would fail to meet the requirement for soundness. If the loss is determined, as permitted, with half-size sieves, the material could be reported as of acceptable quality. Since almost all

Table 4.—Computations of test results for sample tested with Alternate B grading

Size tested (inches)	Actual loss (percent)	Grading as received (percent)	Weighted average loss (percent)
Loss Determined Using Full-Size Sieves			
2–1½	19.4	5	0.97
1½–1	11.8	21	2.48
1–¾	17.2	19	3.27
¾–½	18.1	17	3.08
½–⅜	17.0	16	2.72
⅜–No. 4	11.3	22	2.48
Total		100	15.0
Loss Determined Using Half-Size Sieves			
2–1½	4.8	5	0.24
1½–1	10.9	21	2.29
1–¾	8.1	19	1.54
¾–½	8.9	17	1.51
½–⅜	8.8	16	1.41
⅜–No. 4	7.0	22	1.54
Total		100	8.5

Table 5.—Computation of test results for sample tested with Grading 1

Size tested	Actual loss	Conversion factor	Converted values for loss	Loss for specified combinations of sizes	Grading as received	Weighted average loss
Inches	*Pct.*		*Pct.*	*Pct.*	*Pct.*	*Pct.*
Loss Determined Using Full-Size Sieves						
2–1½	19.4	1.00	19.4	19.4	5	0.97
1½–1	11.2	.67	7.5	13.2	40	5.28
1–¾	17.2	.33	5.7			
¾–½	13.1	.67	8.8	14.4	33	4.75
½–⅜	17.0	.33	5.6			
⅜–No. 4	11.3	1.00	11.3	11.3	22	2.49
Total					100	13.5
Loss Determined Using Half-Size Sieves						
2–1½	4.8	1.00	4.8	4.8	5	0.24
1½–1	9.5	.67	6.4	9.1	40	3.64
1–¾	8.1	.33	2.7			
¾–½	5.4	.67	3.6	6.5	33	2.14
½–⅜	8.8	.33	2.9			
⅜–No. 4	7.0	1.00	7.0	7.0	22	1.54
Total					100	7.6

Table 6.—Computations of test results for sample tested with Alternate A grading

Size tested	Actual loss	Conversion factor	Converted values for loss	Loss for specified combinations of sizes	Grading as received	Weighted average loss
Inches	*Percent*		*Percent*	*Percent*	*Percent*	*Percent*
Loss Determined Using Full-Size Sieves						
2–1½	10.6	0.50	5.3	11.2	26	2.91
1½–1	11.8	.50	5.9			
1–¾	12.0	.67	8.0	14.0	36	5.04
¾–½	18.1	.33	6.0			
½–⅜	8.8	[1].42	3.7	10.2	38	3.87
⅜–No. 4	11.3	[1].58	6.5			
Total					100	11.8
Loss Determined Using Half-Size Sieves						
2–1½	2.3	0.50	1.2	6.6	26	1.72
1½–1	10.9	.50	5.4			
1–¾	5.0	.67	3.4	6.3	36	2.27
¾–½	8.9	.33	2.9			
½–⅜	6.8	[1].42	2.9	7.0	38	2.66
⅜–No. 4	7.0	[1].58	4.1			
Total					100	6.7

[1] Relative amounts of these sizes to be used are not shown in the method of test. Conversion factors used are based on amount of each size in material as received.

Table 7.—Summary of test results for samples tested with three methods of grading

Method	Loss with full-size sieve	Loss with half-size sieve
	Percent	Percent
Grading 1	13.5	7.6
Alternate A	11.8	6.7
Alternate B	15.0	8.5

specifications for aggregate fail to mention which procedure of those available shall be used, it is possible that the acceptance or rejection of a given sample may depend entirely on which one of the six standard methods of test has been used.

Soundness Questionnaire

Reports of cooperative tests made using the 1946 revision of the method indicated that different laboratories failed to obtain a reasonable agreement in the results of tests on the same material. In an effort to determine the cause for this, Subcommittee III-e of Committee C–9, ASTM, sent a questionnaire covering the sulfate soundness test to all laboratories known to use the test. The questionnaire covered in detail the apparatus used and the procedures followed by each laboratory. About 50 laboratories returned the questionnaire. Of these only three stated that all details of the method were followed exactly. Many laboratories indicated departures from the requirements of the specifications in matters which were considered immaterial. Others showed variations which admittedly could influence the test result but which were necessitated by one reason or another including shortage of help in the laboratory. In some respects, the inference could be drawn from replies to the questionnaire that some requirements of the method were too strict, or were unnecessary, and that the operator would conform to these only under protest.

The questionnaire furnished information concerning necessary or desirable changes in the requirements for the method. It was disclosed that many laboratories use pans instead of perforated containers for holding the test samples. Although the use of pans is not permitted in the present method, the only advantage of the use of perforated containers is that the sample can be drained more thoroughly and dried more rapidly. If laboratories have found that samples stored in pans can be brought to oven dryness in a suitable time, and it is presumed that this is so, the requirements of the method should be changed to accept this established practice. A test for the rate of evaporation of the drying ovens was suggested by the writer and furnished some surprising results. The present specifications require that the oven "shall provide a free circulation of air through the oven and shall be capable of maintaining a temperature of 105 to 110° C." The questionnaire requested that a 1,000 ml. beaker containing 500 grams of water at 70° F. be placed in the oven with a group of test samples to be dried, and that the amount of water evaporated in a given time be determined.

Evaporation rate of ovens

Information regarding the rate of evaporation of the oven was furnished by 38 laboratories. The values reported were reduced to grams of water evaporated per hour, and plotted in figure 1 as a frequency distribution curve. The average rate was about 28 grams per hour under the established conditions. The maximum and minimum rates were 61.5 and 1.5 grams per hour. Twenty percent of the ovens had a rate of 22 grams per hour or less, and an equal number had a rate of 45 grams per hour or more. Of 11 ovens of the mechanical convection type, 8 were reported to have an evaporation rate of 28 grams per hour or more.

Significant differences were found between the work done by the various ovens on the basis of the evaporation rate and the reported time normally used for the drying cycle. The computed values for the total amount of water evaporated, assuming the rate remained constant, varied from 16 to 1,080 grams. Only 33 laboratories furnished definite statements of the time normally used during the drying cycle, and the ovens involved are rated as follows:

Grams of water evaporated during drying cycle	*Number of ovens*
Less than 50	1
50–100	4
100–150	10
150–200	3
200–300	8
300–400	3
Over 400	4

It is entirely possible that some of the information from which these values were computed was reported incorrectly. If so, the great differences shown between ovens may be reduced. There were, however, sufficient differences in the rate of evaporation of the various ovens to warrant including in the method, requirements for the performance of the oven. A suitable revision of the requirements for the oven follows. The values included are based on the performance of the ovens reported in the questionnaire, and may need some revision when a larger number of ovens are studied.

> The oven shall be capable of being heated continuously between 105 and 110° C. (221 and 230° F.) and the rate of evaporation of water shall be at least 25 grams per hour. This rate shall be determined by the loss of water from 1-liter, Griffin low form beakers, each containing 500 grams of water at 70° F., placed at each corner and the center of each shelf of the preheated oven, and heated for at least 4 hours during which period the doors of the oven shall be kept closed. The rate of evaporation shall be determined from the average loss for all beakers.

Consideration had been given to the exclusive use of magnesium sulfate in this method because the solubility of this salt is much less variable with change in temperature than that of the sodium salt. With the smaller variation in solubility, a saturated solution of the magnesium salt could more readily be maintained, and duplication of test results should be more certain. In the replies to the questionnaire, it was found that of 33 laboratories which used the method regularly, 21 used sodium sulfate exclusively or as an alternate with the magnesium salt. The fact that almost two-thirds of the reporting laboratories

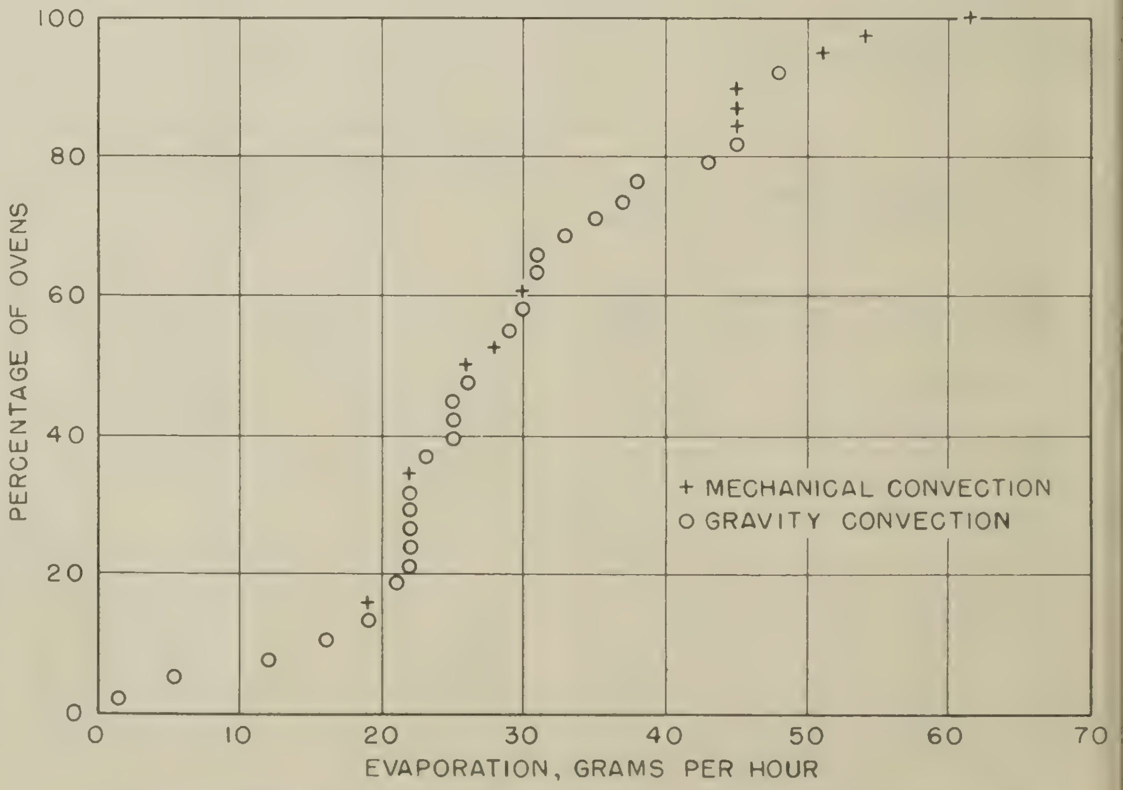

Figure 1.—Rate of evaporation of 38 ovens used in accelerated soundness test.

used sodium sulfate makes it inadvisable to consider its elimination at this time.

Basis for rejection of aggregate

Replies to the questionnaire were received from 23 laboratories which regularly make the test on concrete aggregates proposed for use in construction financed by public funds. Eleven of these laboratories stated that material is rejected on the basis of the sulfate soundness test. The other 12 stated that material is not so rejected, but that the results of freezing and thawing or other tests, with or without service records, are considered. The use of the sulfate soundness test for the arbitrary rejection of aggregate is considered a highly questionable practice. Experience has shown that materials which have a low loss in the test generally have adequate resistance to frost action. It has also been shown that many materials having a high loss in the sulfate soundness test may not resist freezing and thawing, but this is not true of all materials.

A low loss in the soundness test can be associated with resistance to weathering with more assurance than a high loss and lack of resistance to weathering. As an example, a sample of sandstone had a loss of 64 percent in a five-cycle sodium sulfate test. Another portion of the same sample showed a loss of only 2 percent when frozen and thawed 50 times in water. None of the fragments cracked or broke in freezing, and some or all of the loss may have been due to handling. As the stone in question is widely used for building with satisfactory results, it must be concluded that the results of the sulfate soundness test were quite misleading in this case.

It is considered highly advisable to add to the method a definite statement of the applicability of the test result. A suitable revision of the "Scope" of the method could be made by the insertion of one sentence (given in italic) to have the text read as follows:

> This method covers the procedure to be followed in testing aggregates to determine their resistance to disintegration by saturated solutions of sodium sulfate or magnesium sulfate. It furnishes information helpful in judging the soundness of aggregates subject to weathering action, particularly when adequate information is not available from service records of the material exposed to actual weathering conditions. *Generally the results of this test should not be used to reject materials, but aggregates having a low loss in this test can confidently be assumed to be resistant to the effects of freezing and thawing.* Attention is called to the fact that test results by the use of the two salts differ considerably and care must be exercised in fixing proper limits in any specifications which may include requirements for these tests.

Sulfate solution

A number of replies to the questionnaire called attention to two other items which unquestionably have a marked effect on the test result. These are the preparation and maintenance of the sulfate solution. The method requires that a saturated solution of the salt be prepared, and gives quite detailed instructions regarding the procedure to be followed. This does not appear to be a formidable undertaking, nor does it appear difficult to maintain that solution in its saturated condition provided the temperature of the solution can be controlled within the specified limits of 68 to 72° F.

Comment was made that the present requirements in the method call for an excessive amount of the sulfate. Use of the amount indicated was found to furnish a solid crystalline mass of the sulfate in the container for the saturated solution. As the solid crystal does not enter solution readily, it is quite possible for the liquid in the container to become less than saturated even in the presence of this crystalline mass. The method of test now requires that the solution shall be stirred thoroughly immediately before use. It is doubted that this will cause material from the crystalline mass to enter solution unless the mass is broken to fine size. Consequently it is recommended that the method be revised to require breaking of the mass prior to agitation. The addition of a suitable statement to the note concerning the amount of salt to be used appears desirable.

Half-Size Sieves and the Reverse Cycle

The use of a sieve having openings one-half the size of those in the sieve used to prepare the sample has already been mentioned. The loss through the half-size sieve should be a better indication of the soundness of a material than that passing the initial retaining sieve. When the latter sieve is used, some otherwise sound fragments may pass the sieve due to minor chipping or breaking of corners or edges. Although these chips would probably pass a half-size sieve, any considerable amount of material passing this sieve would be truly unsound, having suffered multiple cracking or marked disintegration. With elimination of the accidental inclusion of sound fragments in the loss, more concordant results between laboratories should be obtained.

In the present method, the sample is immersed in the sulfate solution for 16 to 18 hours, and then dried to constant weight. Most laboratories endeavor to maintain a 24-hour cycle, and limit the drying period to 5 to 7 hours. As it has been shown that this period of drying may not be sufficient to dry the sample thoroughly, consideration was given to the use of a reverse cycle method. In this, the sample was immersed in the sulfate solution for about 6 hours, and dried in the oven for 16 to 17 hours. Previous work had indicated that coarse aggregate would absorb as much water in 6 hours as it would in 24 hours, and that a drying period of the length shown would dry soundness samples even in ovens with a relatively low rate of evaporation. As thorough drying of the sample is necessary for the maximum crystallization of the sulfate salt, it was believed that the most effective results and the desired 24-hour cycle could be obtained with the reverse cycle method.

Tests of coarse aggregate were made to determine whether the use of half-size sieves or the reverse cycle would furnish more concordant results. Nine samples of stone of high to low loss were selected to determine the suitability of the procedures mentioned above. Each sample was crushed and screened to the 1½- to 1-inch size. Six test samples were prepared from each stone to make a total of 54 samples tested. Three samples of each material were subjected to five repetitions of the standard cycle consisting of an immersion period of 17 hours and a drying period of 6 hours. An equal number of sam-

Table 8.—Tests by standard and reverse cycle methods on 1½- to 1-inch stone

Sample number	Loss passing full-size sieve				Loss passing half-size sieve			
	Test 1	Test 2	Test 3	Average	Test 1	Test 2	Test 3	Average
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
STANDARD CYCLE								
89805	45.0	39.2	45.6	43.3	20.5	16.9	15.9	17.8
89812	19.9	32.4	8.1	20.1	8.7	11.6	3.0	7.8
89813	17.2	19.5	33.3	23.3	6.8	5.6	8.2	6.9
89889	8.0	6.2	2.0	5.4	4.5	4.5	1.4	3.5
89890	74.0	84.2	77.2	78.5	51.7	55.6	57.5	54.9
89892	10.1	4.2	4.1	6.1	3.9	1.5	3.6	3.0
89916	13.4	6.0	14.1	11.2	3.8	.5	4.8	3.0
89918	14.2	7.0	32.7	18.0	6.5	2.1	16.5	8.4
89920	40.2	37.5	36.7	38.1	7.1	8.8	12.6	9.5
REVERSE CYCLE								
89805	41.9	28.7	56.4	42.3	17.3	9.3	23.2	16.6
89812	7.3	15.1	4.0	8.8	2.3	5.3	2.0	3.2
89813	12.5	17.7	22.2	17.5	5.4	6.4	9.8	7.2
89889	2.8	4.3	4.6	3.9	2.8	2.6	4.1	3.2
89890	85.3	53.4	94.5	77.7	55.1	34.6	68.9	52.9
89892	12.7	6.1	12.9	10.6	5.4	3.6	4.7	4.6
89916	.8	6.5	8.6	5.3	.1	1.9	2.2	1.4
89918	19.6	11.0	14.5	15.0	9.8	6.3	6.2	7.4
89920	18.5	8.8	19.2	15.5	4.9	.3	5.5	3.6

Table 9.—Average values and mean deviations from average values for tests by standard and reverse cycle methods

Method	Sieve used to determine loss	Average loss, all samples	Mean deviation from average loss
		Percent	*Percent*
Standard cycle	Full-size	27.1	28.2
Do	Half-size	12.8	31.5
Reverse cycle	Full-size	21.8	29.7
Do	Half-size	11.1	33.0

ples were subjected to the reverse cycle with an immersion period of 6 hours and a drying period of 17 hours. At the completion of the fifth cycle, each sample was washed, dried, and sieved to refusal on the 1- and ½-inch sieves. The results obtained are shown in table 8.

Variation in test results

It will be observed that some test results differ to a marked extent from companion results for the same material. This is found in tests by each method. Computation of a mean deviation from the average for all results obtained by each method of treatment and each method of determining the loss furnished very nearly the same value for each of the four groups of results. This indicates that nonuniformity of the materials tested probably is responsible for the variations found in some of the test results. It is believed that the effect of the wild results will be cancelled out if all results obtained by each of the four methods of test are considered, and that comparisons between average values for each method are valid. These average values as well as values for mean deviations from the average are shown in table 9.

It was expected that the reverse cycle would insure more thorough drying of the samples than would be obtained with the standard method. Due to this, it was believed that the reverse cycle would furnish a greater loss for each material with more uniform results from sample to sample. These were not obtained because the oven used had a sufficient rate of evaporation to dry the samples in the 6-hour period provided in the standard method.

Several matters of interest are found in the results obtained in these tests. The smaller losses for the reverse cycle method indicate that insufficient time was allowed for either the absorption of the sulfate solution or the crystallization of the salt, or both. The better agreement between the average losses determined with the half-size sieve than those for the full-size sieve indicates that with reduction in size of the fragment tested, the time allowed in the reverse cycle for absorption or crystallization becomes less critical.

Absorption rate of sulfate solution

These results are further interpreted to indicate that the factor controlling the loss in the reverse cycle is the time for absorption of the sulfate solution. The rate of crystallization would be the same irrespective of the method used, but the rate of absorption would vary inversely with the size of fragment.

The rate of absorption of the sulfate solution may also be affected if the mouth of the capillary tube or crack in the fragment is plugged by the sulfate salt. The salt carried into the rock during each immersion period tends to be drawn toward the surface of the rock during the drying period and forms a deposit at or near the surface. During subsequent immersion periods, the solution must pass this plug. As the solution is saturated, the plug of salt cannot be dissolved but must be softened until the solution can push past or around it. Considerable time probably is required for this and the 6-hour immersion period in the reverse cycle may be insufficient to permit complete saturation of the larger sizes of coarse aggregate.

It is believed that with the adoption of requirements for the oven used in this method of test, samples may be dried in a period of 6 hours and use of a reverse cycle will be unnecessary. The use of the half-size sieve in the determination of the loss of samples of coarse aggregate is recommended.

Size of Sample

Coarse aggregate

For a coarse aggregate graded from 2 inches to the No. 4 sieve, the present method requires a test sample weighing at least 4,800 to 6,050 grams, depending upon the grading chosen for the material tested. This material is separated into 3 to 6 portions also depending on the grading to be used. If several samples

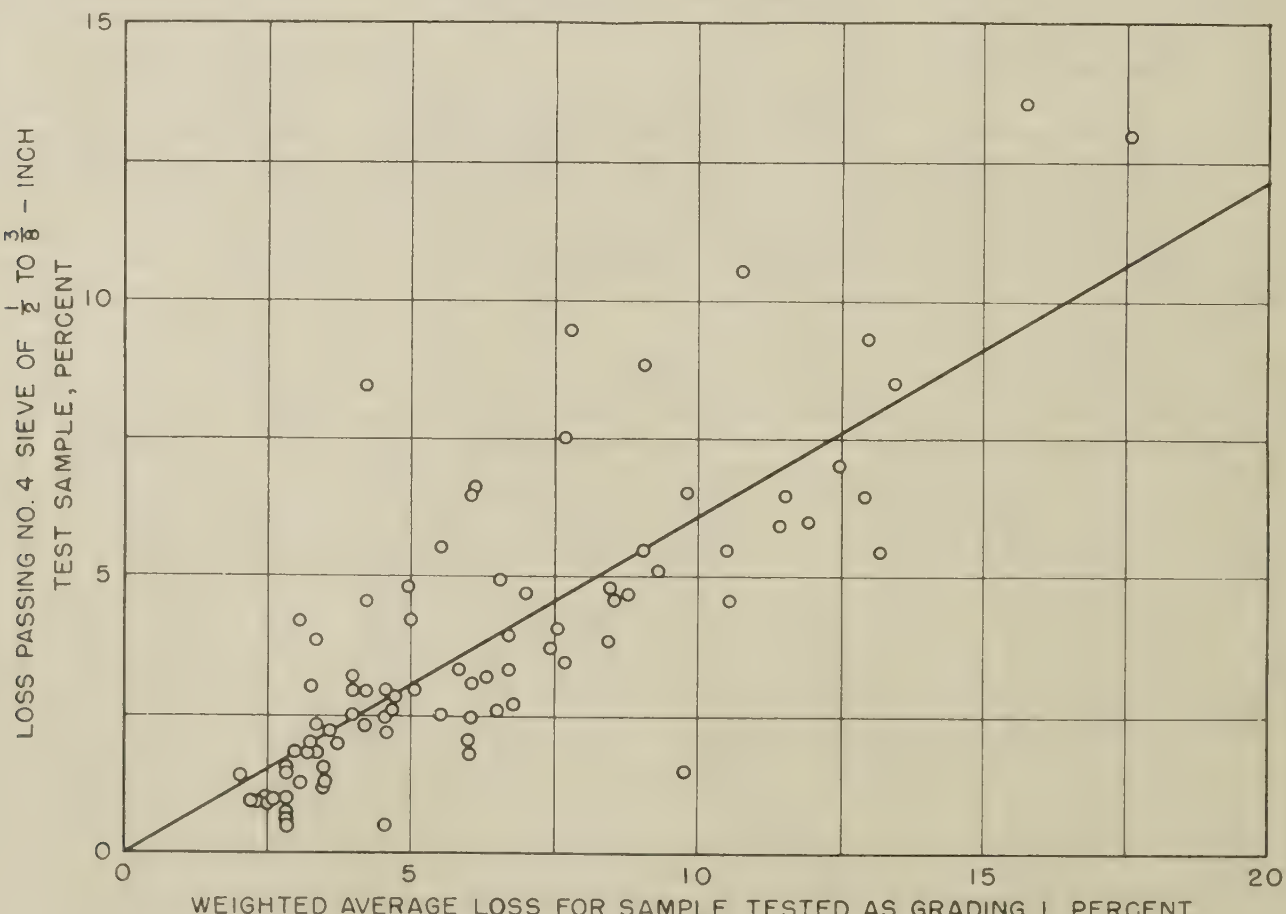

Figure 2.—Comparison of losses for coarse aggregate.

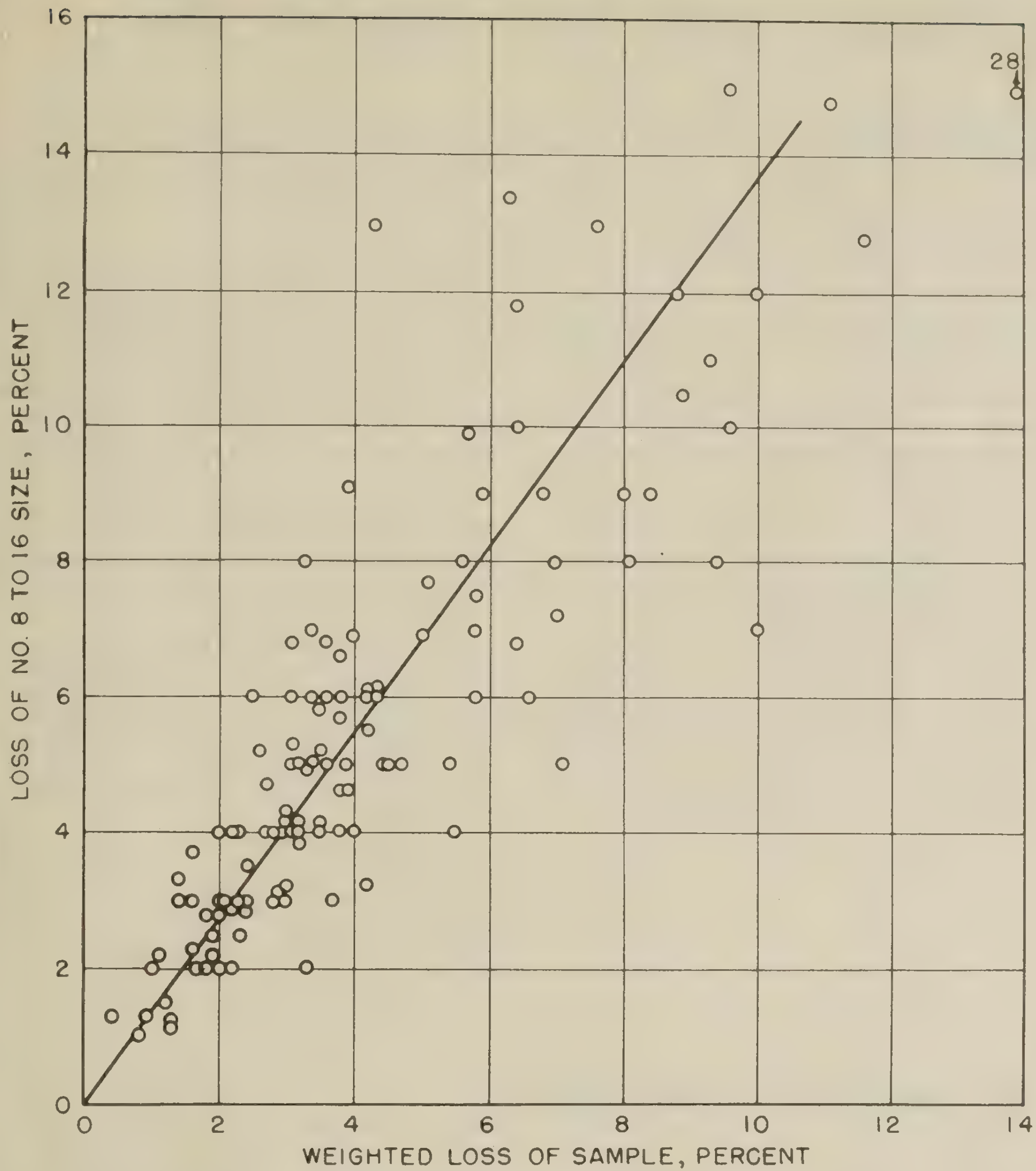

Figure 3.—Comparison of losses for fine aggregate.

are tested simultaneously, the total amount of material under test and the number of containers used may tax the facilities of the laboratory. In laboratories with limited personnel, it has been observed that there is a marked tendency for reduction in the amount of material tested.

Although this reduction may have little or no effect on materials of uniform composition, it might tend to prevent reliable test results on nonhomogeneous materials. To determine whether any change could be made in the method which would permit the use of a smaller sample and reduce the labor and attention required, the results of tests of over 250 samples were studied. These samples had been tested in the laboratory of the Bureau of Public Roads during the period 1945–53.

Most of the samples of coarse aggregate had a maximum size of 1½ or 2 inches, and most had been tested by individual size of fragment, the Alternate B method. The results were studied to determine whether the number of sizes of aggregate could be reduced and still obtain a loss approximating that for all sizes tested. Comparison of the averages for the entire group of samples indicated that the loss for the ½- to ⅜-inch size agreed closely with the loss for all sizes tested. From these data, it would appear that a good knowledge of the soundness of a material may be obtained by test of the ½- to ⅜-inch size alone. As a test of this size would require a test sample weighing only 500 grams, a great reduction in the weight of material and number of samples tested would result.

Losses through both the full-size and half-size sieves had been determined for about 150 of the samples tested by the Alternate B method. To determine the effect on the specification requirement of a desirable change in the method of testing, figure 2 was prepared from data regarding these samples. This shows a comparison between the weighted average loss through the original retaining sieve of samples of 1½- or 2-inch maximum size coarse aggregate when tested as required for Grading 1, and the loss of ½- to ⅜-inch aggregate passing a half-size sieve. Samples which had a weighted average loss of less than 2 percent were not plotted, nor were several samples with quite high losses.

ASTM Specification C 33 and AASHO Specification M 80 permit the acceptance of coarse aggregate for concrete having a loss in the sodium sulfate soundness test of 12 percent. As both specifications are silent regarding the method for determining this loss, the method considered here is that involving the use of Grading 1 with the loss passing the original retaining or full-size sieve. Figure 2 shows that material of the quality indicated will have a loss of 7 percent passing the half-size (No. 4) sieve when tested as ½- to ⅜-inch material. It is believed that sufficient data have been presented to insure the reliability of this conversion in specification requirements.

Fine aggregate

A different problem was found in connection with the most desirable method for testing fine aggregate, especially natural sand. As a general rule, the loss of natural sand in a soundness test varies directly with the size of the sand grain. As the size of grain is reduced, the percentage of single grains of quartz increases, and these grains have very little loss in a soundness test. It is believed that the testing for soundness of fine grains of natural sand is largely wasted effort, and that a large loss found in tests of such material probably resulted from accidental loss of some of the sample.

However, an adequate knowledge of the soundness of fine aggregate can be obtained by test of a single size of grain. The selection of the size tested must be based on the availability of that size in sand as customarily used, and on the occurrence in that size of the more unsound grains. It is believed that the No. 8 to No. 16 size best meets these requirements. Although a coarser size may contain more unsound grains, many sands have very little material retained on the No. 8 sieve, and the use of this coarser size cannot be considered.

A comparison between the average loss weighted with respect to the grading of the sample as required by the present method of test and the loss for the No. 8 to No. 16 size, is shown in figure 3. The loss in each case was determined from material passing the original retaining sieve. Approximately 125 samples of sand are represented. The loss for the single size is slightly greater than the weighted average for all sizes. A sand having an average loss for all sizes of 10 percent would have a loss of about 13.5 percent for the No. 8 to No. 16 size. Established specification limits for fine aggregate can readily be revised to make provision for this difference.

Annual Report of the Bureau of Public Roads, Fiscal Year 1955

The *Annual Report of the Bureau of Public Roads* for fiscal year 1955 is now available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 25 cents a copy.

Reflecting the prime significance of highway transport, the report covers a wide range of engineering, administrative, and research activities in this field. It discusses all phases of the Federal-aid construction program, which reached new high levels in 1955. Included are significant improvements on the National System of Interstate Highways as well as developments on other primary and urban highways and farm-to-market roads.

To accelerate the highway improvement program, the Federal-aid authorization of $875 million for the fiscal year 1956 was apportioned to the States on July 1, 1954, 6 months ahead of the time limit set by Congress.

During the fiscal year, $671 million of Federal funds were used in the construction of 22,155 miles of highways with a total cost of $1,280 million. Included were 6,050 miles of highways and 1,202 bridges on the Federal-aid primary system outside of cities, 842 miles of highways and 465 bridges on urban portions of the primary system, 14,692 miles of roads and 1,764 bridges on secondary roads, and 571 miles of highways in National parks, forests, parkways, and on flood-relief projects. Railway-highway grade crossings were eliminated at 216 locations, and 317 crossings were protected by installation of appropriate safety devices.

A number of outstanding Federal-aid projects are described in the report, which also reviews factors affecting progress, new highway legislation, foreign activities including improvements on the Inter-American Highway, and work in the field of traffic safety.

Urban problems received particular attention. As the report shows, in the selecting of city projects for Federal aid every effort was made to eliminate traffic bottlenecks by construction that provided greater traffic capacity. Expressways were under construction in more than 100 cities. Construction of circumferential routes near the outskirts of metropolitan areas was started in Boston, Louisville, Baltimore, and other centers. This will permit a free exchange of traffic between radial routes without passing through the more congested areas. Great stress was laid on the need for full control of access on all arterial highways.

Four special reports were submitted to Congress by the Bureau, among them the most comprehensive study of highway needs ever undertaken. This report, prepared in cooperation with the State highway departments, showed that the cost of construction required to modernize the Nation's roads and streets in a period of 10 years would be $101 billion.

In the field of research, the annual report covers Bureau studies of highway finance, highway transport, and the array of physical problems that are associated with highway construction.

Reviewing other significant developments, the report points out that the record amount of highway construction during the year was overshadowed in the public mind by the need for a much greater volume of highway improvements to reduce congestion, accidents, and delays.

The Identification of Rock Types

To meet popular demand, a convenient 6 x 9-inch reprint has again been made of the article *The Identification of Rock Types*, by D. O. Woolf, which appeared in Public Roads, vol. 26, No. 2, June 1950. The article presents a simple method for use by the highway engineer in making field identification of the different types of rock with which he is concerned. It is extremely useful to engineers, engineering students, and others whose work requires a limited, practical knowledge of geology. The reprint is for sale by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 15 cents a copy.

U. S. GOVERNMENT PRINTING OFFICE: 1956

A list of the more important articles in Public Roads may be obtained upon request addressed to Bureau of Public Roads, Washington 25, D. C.

PUBLICATIONS of the Bureau of Public Roads

The following publications are sold by the Superintendent of Documents, Government Printing Office, Washington 25, D. C. Orders should be sent direct to the Superintendent of Documents. Prepayment is required.

ANNUAL REPORTS

Work of the Public Roads Administration:
- 1941, 15 cents. 1948, 20 cents.
- 1942, 10 cents. 1949, 25 cents.

Public Roads Administration Annual Reports:
- 1943; 1944; 1945; 1946; 1947.
- (*Free from Bureau of Public Roads*)

Annual Reports of the Bureau of Public Roads:
- 1950, 25 cents. 1952, 25 cents. 1954 (out of print).
- 1951, 35 cents. 1953, 25 cents. 1955, 25 cents.

PUBLICATIONS

Bibliography of Highway Planning Reports (1950). 30 cents.
Braking Performance of Motor Vehicles (1954). 55 cents.
Construction of Private Driveways, No. 272MP (1937). 15 cents.
Criteria for Prestressed Concrete Bridges (1954). 15 cents.
Design Capacity Charts for Signalized Street and Highway Intersections (reprint from Public Roads, Feb. 1951). 25 cents.
Electrical Equipment on Movable Bridges, No. 265T (1931). 40 cents.
Factual Discussion of Motortruck Operation, Regulation, and Taxation (1951). 30 cents.
Federal Legislation and Regulations Relating to Highway Construction (1948). Out of print.
Financing of Highways by Counties and Local Rural Governments: 1931–41, 45 cents; 1942–51, 75 cents.
General Location of the National System of Interstate Highways, Including All Additional Routes at Urban Areas Designated in September 1955. 55 cents.
Highway Bond Calculations (1936). 10 cents.
Highway Bridge Location No. 1486D (1927). 15 cents.
Highway Capacity Manual (1950). $1.00.
Highway Needs of the National Defense, House Document No. 249 (1949). 50 cents.
Highway Practice in the United States of America (1949). 75 cents.
Highway Statistics (annual):

1945 (out of print).	1948, 65 cents.	1951, 60 cents.
1946, 50 cents.	1949, 55 cents.	1952, 75 cents.
1947, 45 cents.	1950 (out of print).	1953, $1.00.

Highway Statistics, Summary to 1945. 40 cents.
Highways in the United States, *nontechnical* (1954). 20 cents.
Highways of History (1939). 25 cents.
Identification of Rock Types (reprint from Public Roads, June 1950). 15 cents.
Interregional Highways, House Document No. 379 (1944). 75 cents.
Legal Aspects of Controlling Highway Access (1945). 15 cents.
Local Rural Road Problem (1950). 20 cents.
Manual on Uniform Traffic Control Devices for Streets and Highways (1948) (*including 1954 revisions supplement*). $1.00.
  Revisions to the Manual on Uniform Traffic Control Devices for Streets and Highways (1954). *Separate*, 15 cents.
Mathematical Theory of Vibration in Suspension Bridges (1950). $1.25.
Model Traffic Ordinance (revised 1953). Out of print.

PUBLICATIONS (Continued)

Needs of the Highway Systems, 1955–84, House Document No. 120 (1955). 15 cents.
Opportunities in the Bureau of Public Roads for Young Engineers (1955). 25 cents.
Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft (1943). $2.00.
Progress and Feasibility of Toll Roads and Their Relation to the Federal-Aid Program, House Document No. 139 (1955). 15 cents.
Public Control of Highway Access and Roadside Development (1947). 35 cents.
Public Land Acquisition for Highway Purposes (1943). 10 cents.
Public Utility Relocation Incident to Highway Improvement, House Document No. 127 (1955). 25 cents.
Results of Physical Tests of Road-Building Aggregate (1953). $1.00.
Roadside Improvement, No. 191MP (1934). 10 cents.
Selected Bibliography on Highway Finance (1951). 60 cents.
Specifications for Construction of Roads and Bridges in National Forests and National Parks, FP-41 (1948). $1.50.
Standard Plans for Highway Bridge Superstructures (1953). $1.25.
Taxation of Motor Vehicles in 1932. 35 cents.
Tire Wear and Tire Failures on Various Road Surfaces (1943). 10 cents.
Transition Curves for Highways (1940). $1.75.

MAPS

State Transportation Map series (available for 39 States). Uniform sheets 26 by 36 inches, scale 1 inch equals 4 miles. Shows in colors Federal-aid and State highways with surface types, principal connecting roads, railroads, airports, waterways, National and State forests, parks, and other reservations. Prices and number of sheets for each State vary—see Superintendent of Documents price list 53.
United States System of Numbered Highways together with the Federal-Aid Highway System (also shows in color National forests, parks, and other reservations). 5 by 7 feet (in 2 sheets), scale 1 inch equals 37 miles. $1.25.
United States System of Numbered Highways. 28 by 42 inches, scale 1 inch equals 78 miles. 20 cents.

Single copies of the following publications are available to highway engineers and administrators for official use, and may be obtained by those so qualified upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

Bibliography on Automobile Parking in the United States (1946).
Bibliography on Highway Lighting (1937).
Bibliography on Highway Safety (1938).
Bibliography on Land Acquisition for Public Roads (1947).
Bibliography on Roadside Control (1949).
Express Highways in the United States: a Bibliography (1945).
Indexes to Public Roads, volumes 17–19 and 23.
Title Sheets for Public Roads, volumes 24–27.

UNITED STATES
GOVERNMENT PRINTING OFFICE
DIVISION OF PUBLIC DOCUMENTS
WASHINGTON 25, D. C.

OFFICIAL BUSINESS

DEPARTMENT OF COMMERCE - BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF DECEMBER 31, 1955

FOR OFFICIAL DISTRIBUTION

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$11,324	$20,141	$11,126	356.3	$7,579	$4,024	59.8	$35,759	$18,696	447.7	$63,479	$33,846	863.8
Arizona	3,937	9,386	6,957	171.2	2,476	1,896	38.4	9,016	6,587	118.9	20,878	15,440	328.5
Arkansas	14,368	9,878	5,271	311.5	4,375	2,172	45.0	20,962	10,601	414.0	35,215	18,044	770.5
California	11,007	33,812	18,537	201.6	25,025	13,016	23.9	131,906	67,462	276.5	190,743	99,015	502.0
Colorado	17,617	7,489	4,158	90.1	2,827	1,701	25.8	13,662	7,338	148.3	23,978	13,197	264.2
Connecticut	22,659	1,760	985	5.2	366	182	.3	8,261	4,107	12.4	10,387	5,274	17.9
Delaware	6,006	2,079	1,055	9.6	1,607	828	29.1	4,113	2,061	21.7	7,799	3,944	60.4
Florida	11,003	13,096	7,031	219.5	16,626	8,675	76.3	29,561	15,070	307.1	59,283	30,776	602.9
Georgia	23,503	24,294	12,591	434.2	9,441	4,316	68.6	39,108	18,520	697.2	72,843	35,427	1,200.0
Idaho	8,298	4,482	2,965	61.5	2,920	2,070	44.8	11,980	7,716	155.2	19,382	12,751	261.5
Illinois	21,136	54,038	29,365	498.3	15,460	8,326	31.7	82,503	44,397	466.6	152,001	82,088	996.6
Indiana	31,106	16,294	8,349	71.4	18,223	10,148	95.3	36,002	19,244	80.4	70,519	37,741	247.1
Iowa	14,289	20,000	11,326	517.9	8,245	4,253	118.9	18,519	9,858	825.7	46,764	25,437	1,462.5
Kansas	13,781	11,176	5,865	628.1	7,874	4,159	130.5	22,957	11,744	870.1	42,007	21,768	1,628.7
Kentucky	15,335	4,738	2,452	19.0	6,273	3,862	13.4	36,382	18,773	601.2	47,393	25,087	633.6
Louisiana	13,547	11,163	5,582	108.0	4,214	2,107	12.9	45,793	21,777	513.4	61,170	29,466	634.3
Maine	7,993	7,688	4,124	55.9	650	286	1.4	10,048	5,151	68.0	18,386	9,561	125.3
Maryland	8,740	23,603	12,065	58.8	8,437	4,144	8.1	15,900	8,435	64.5	47,940	24,644	131.4
Massachusetts	13,658	23,663	12,501	32.9	7,578	3,767	6.5	46,669	22,429	55.0	77,910	38,697	94.4
Michigan	17,090	40,530	20,976	481.4	23,625	13,183	100.7	43,310	21,516	321.1	107,465	55,675	903.2
Minnesota	15,437	18,907	9,537	576.1	2,389	1,627	73.2	26,795	14,253	476.3	48,091	25,417	1,125.6
Mississippi	14,633	11,102	5,450	390.9	1,998	1,074	39.9	26,004	13,449	611.4	39,104	19,973	1,042.2
Missouri	8,787	34,082	17,421	1,020.4	7,835	4,075	24.3	75,604	39,716	1,256.8	117,521	61,212	2,301.5
Montana	17,042	9,572	5,897	172.2	4,874	2,822	91.6	22,913	14,391	387.8	37,359	23,110	651.6
Nebraska	14,904	19,328	10,152	853.0	7,725	3,579	89.5	22,900	12,308	719.1	49,953	26,039	1,661.6
Nevada	11,852	4,610	3,898	75.0	1,718	1,437	30.9	4,452	3,790	88.4	10,780	9,125	194.3
New Hampshire	6,978	2,572	1,294	16.0	514	257	2.6	7,311	3,782	45.9	10,397	5,333	64.5
New Jersey	26,099	9,741	4,339	53.4	4,715	2,354	5.3	28,596	13,324	43.2	43,052	20,017	101.9
New Mexico	9,441	4,188	2,643	50.8	3,187	2,028	57.2	9,442	6,251	181.7	16,817	10,922	289.7
New York	55,352	37,956	19,566	60.5	39,369	20,236	83.5	213,384	98,504	255.7	290,709	138,306	399.7
North Carolina	17,343	22,894	11,381	463.3	5,809	2,902	44.5	39,754	19,951	504.2	68,457	34,234	1,012.0
North Dakota	8,503	12,357	6,249	1,259.1	3,705	1,893	317.4	8,937	4,571	500.6	24,999	12,713	2,077.1
Ohio	28,497	41,720	21,458	147.6	29,373	15,917	54.5	64,124	30,242	84.0	135,217	67,617	286.1
Oklahoma	15,922	18,966	9,651	356.3	12,974	7,043	79.7	32,103	16,690	357.8	64,043	33,384	793.8
Oregon	5,610	6,411	3,604	55.6	5,378	3,317	32.1	19,239	12,038	203.8	31,028	18,959	291.5
Pennsylvania	34,421	14,318	7,592	44.7	64,041	32,695	76.7	78,627	39,925	291.2	156,986	80,212	412.6
Rhode Island	2,479	2,358	1,179	12.5	7,815	4,221	7.4	13,135	6,580	19.9	23,308	11,980	39.8
South Carolina	13,360	14,901	8,097	352.6	2,222	1,148	21.1	17,965	9,506	297.0	35,088	18,751	670.7
South Dakota	3,477	22,547	13,002	822.1	4,474	2,681	137.5	12,236	7,023	528.2	39,257	22,706	1,487.8
Tennessee	21,034	19,217	9,570	263.8	8,954	4,479	78.9	35,233	15,853	254.4	63,404	29,902	597.1
Texas	33,606	17,114	8,764	408.6	37,988	19,486	267.7	90,042	47,633	1,368.1	145,144	75,883	2,044.4
Utah	7,405	2,166	1,619	35.7	4,899	3,689	27.2	9,761	7,481	123.6	16,826	12,789	186.5
Vermont	4,543	3,361	1,683	15.2	479	239	1.6	8,259	4,321	87.2	12,099	6,243	104.0
Virginia	18,664	17,332	8,758	270.7	4,889	2,645	22.8	23,622	11,514	247.6	45,843	22,917	541.1
Washington	13,077	15,264	8,335	221.1	3,341	1,942	48.9	22,671	11,868	128.5	41,276	22,145	398.5
West Virginia	17,521	9,552	5,019	39.6	2,235	1,132	4.1	17,516	8,794	82.3	29,303	14,945	126.0
Wisconsin	18,646	14,679	7,772	137.4	3,301	1,732	43.1	44,818	22,348	392.7	62,798	31,852	573.2
Wyoming	3,673	8,353	5,462	163.4	242	156	10.1	13,252	8,695	237.6	21,847	14,313	411.1
Hawaii	5,065	1,830	915	3.1	4,503	2,233	5.9	4,385	1,972	11.0	10,718	5,120	20.0
District of Columbia	9,769	4,944	2,472	3.7	434	217	.9	8,875	3,965	2.7	14,253	6,654	7.3
Puerto Rico	11,909	4,533	2,203	25.9				17,350	7,976	63.4	21,883	10,179	89.3
TOTAL	761,446	766,185	408,263	12,702.7	455,201	242,371	2,711.5	1,681,716	860,196	16,317.1	2,903,102	1,510,830	31,731.3

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