HYDROLOGY L16/D3 NO PLANS

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No PLANS

EARTH ENVIRONMENTAL CONSULTANTS INCORPORATED

3208 Richmond, N.E. • Albuquerque, New Mexico 87107 • Telephone (505) ~~344-7980~~ 345-6487

May 17, 1976

Mr. Ron Peters
Burns & Peters, Architects
8100 Mountain Road N. E.
Albuquerque, New Mexico 87106

Dear Mr. Peters:

Transmitted herewith are two copies of the site-drainage plan, prepared for the site of the three 8-plex units, to be constructed by Young America Homes at Thaxton and Girard S. E.

Should there be questions regarding our report, please contact us.

Sincerely yours,

EARTH ENVIRONMENTAL CONSULTANTS, INC.

T. C. Patterson, P.E.

TCP/cw
Copy to: Young America Homes w/report

SITE DRAINAGE PLAN

PROPOSED APARTMENT COMPLEX - YOUNG AMERICA HOMES
AT THAXTON AND GIRARD, S. E.
ALBUQUERQUE, NEW MEXICO

PREPARED BY

EARTH ENVIRONMENTAL CONSULTANTS, INC.

FOR

BURNS & PETERS, ARCHITECTS

AND

YOUNG AMERICA HOMES

MAY, 1976

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I. LOCATION AND PRESENT DRAINAGE CONDITIONS

The proposed construction of three, two-story, eight-plex apartment units is to be on lots 1, 2, and 3 of Block 5, and lot 4 of Block 5-A, of the Loma Linda Subdivision. The northern boundary of lot 1 is adjacent to Thaxton, S.E., while all four lots front on Girard, S.E. The general location is shown in FIGURE 1.

The site (four lots) slopes from northeast to southwest at about 5.5 percent. Total drop from the northeast corner to the southwest corner is fifteen feet. Existing one foot contours are shown in FIGURE 2. The topographic information was generated through a survey conducted by the Hall Surveying Company of Albuquerque.

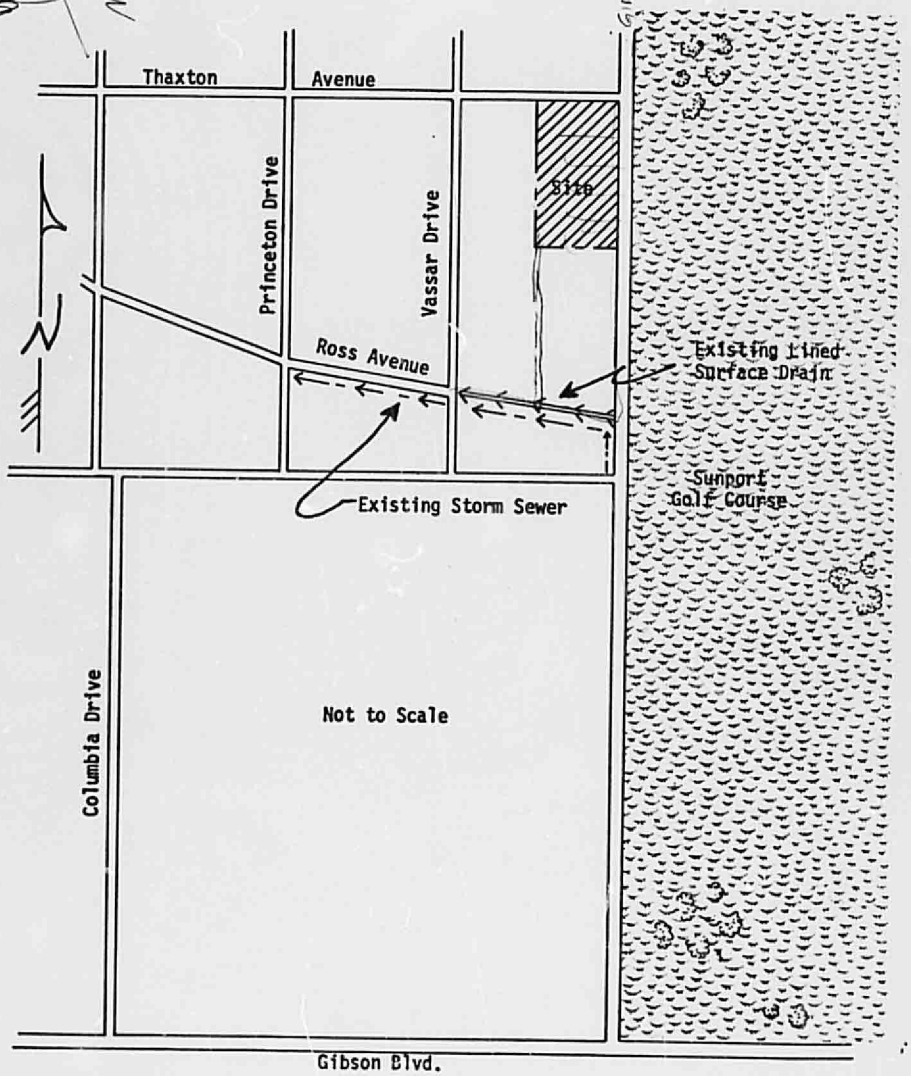
Surface soils (0-4') at the site appear quite uniform and would be classified as loamy sands in the USDA classification system and either SM or SP in the Unified Soil Classification System. Investigation of soil surface conditions was supplemented by two shallow borings (location shown on FIGURE 2). In boring #1, a well permeameter test using U.S. Bureau of Reclamation procedures was conducted to define the hydraulic conductivity (or permeability) of the soil at the site. Details of this test and the results are shown in FIGURE 3.

Present vegetation at the site is very sparse, with few perennials. Considering slope, soils, and vegetation, a "C" value or runoff coefficient of .35 was selected for evaluating current rainfall-runoff relationships using the "rational" method.

In evaluating site drainage, the Albuquerque Metropolitan Arroyo Flood Control Authority, in their 1972 Resolution, established the Project Storm as a 100 year, six-hour rainfall. On very small areas, peak runoff rates are produced by high intensity storms of short duration.

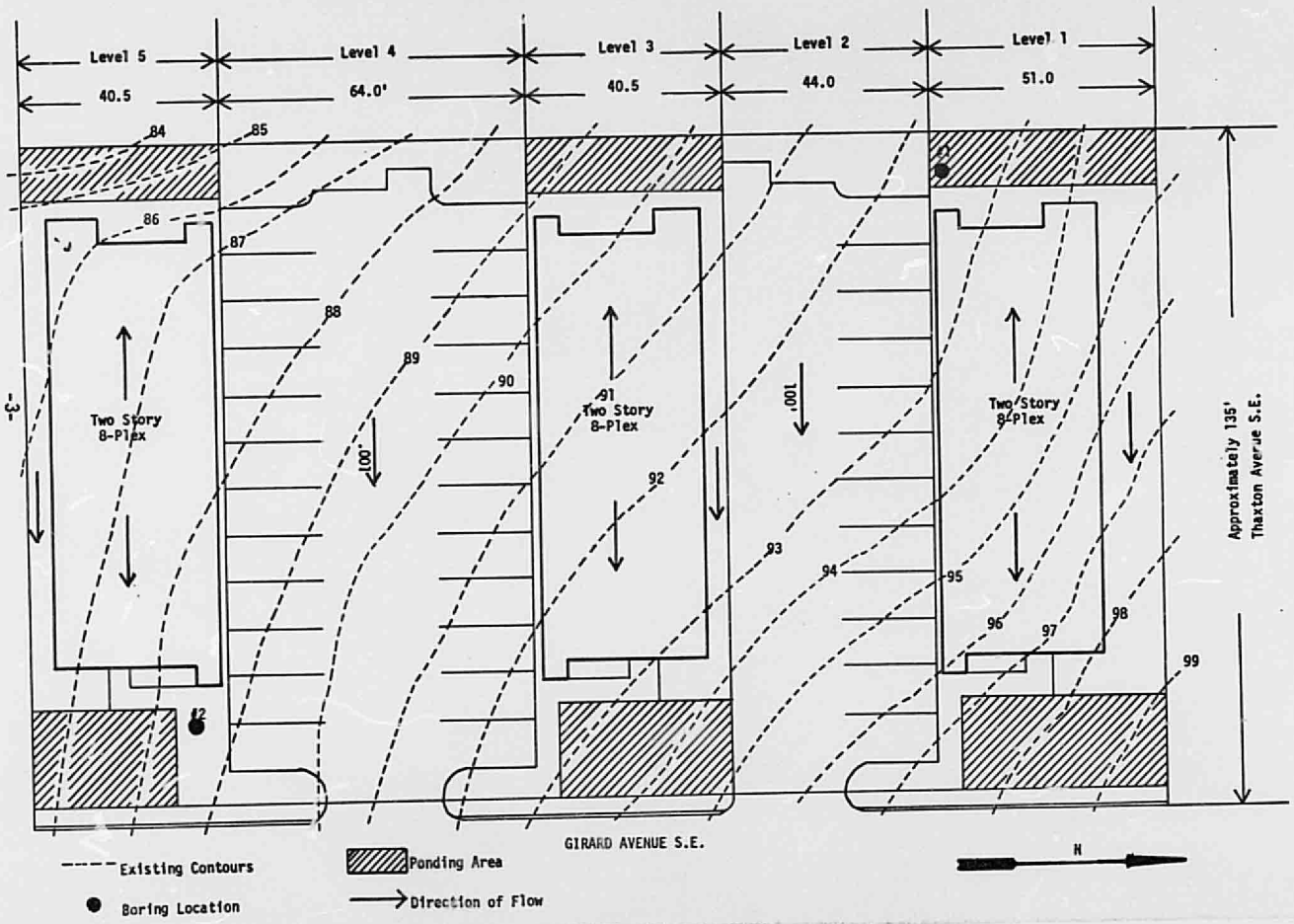
FIGURE 1

SKETCH SHOWING GENERAL LOCATION OF SITE



RE 2

SITE PLAN SHOWING PLANNED SITE DRAINAGE



URE 2

SITE PLAN SHOWING PLANNED SITE DRAINAGE

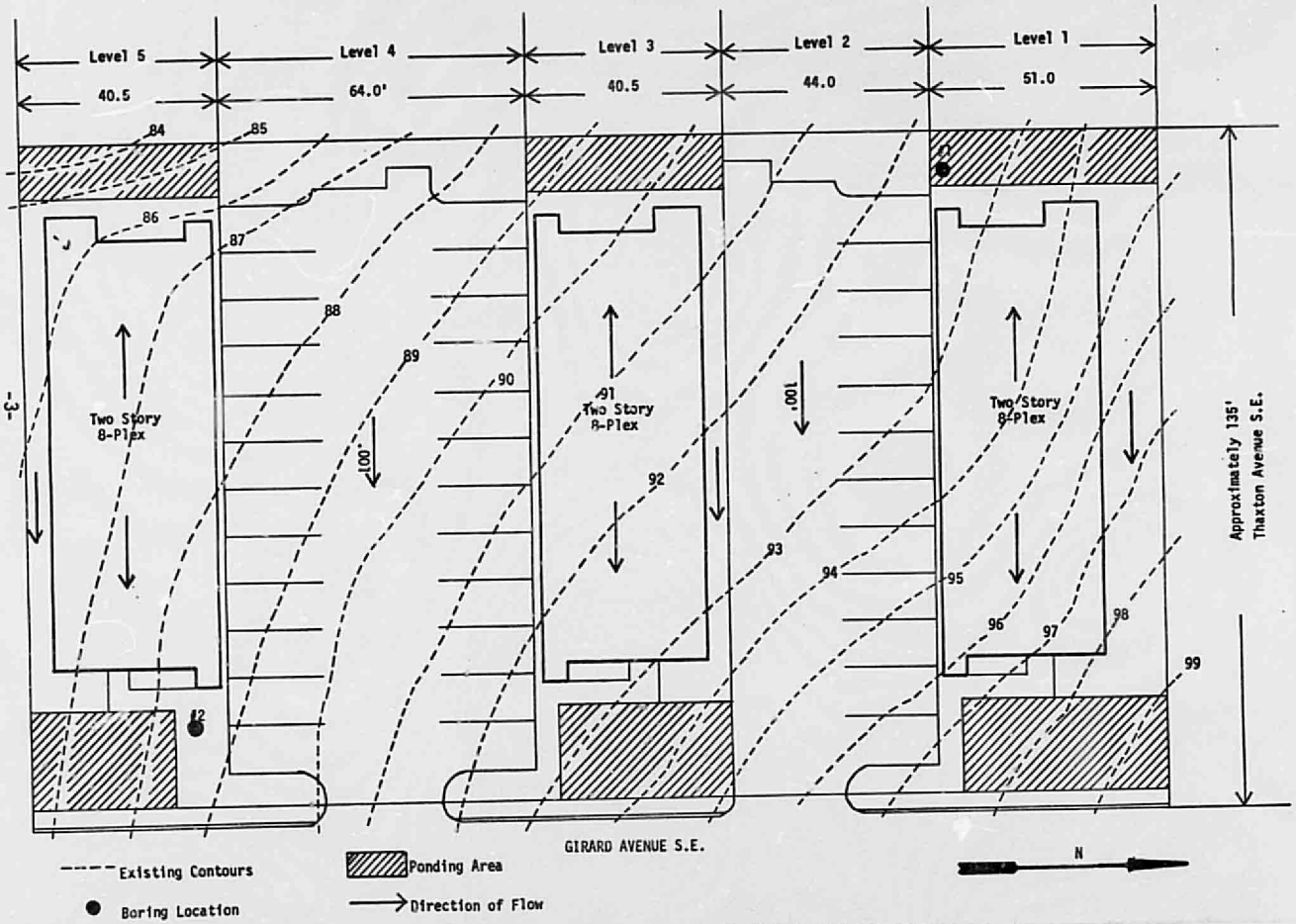
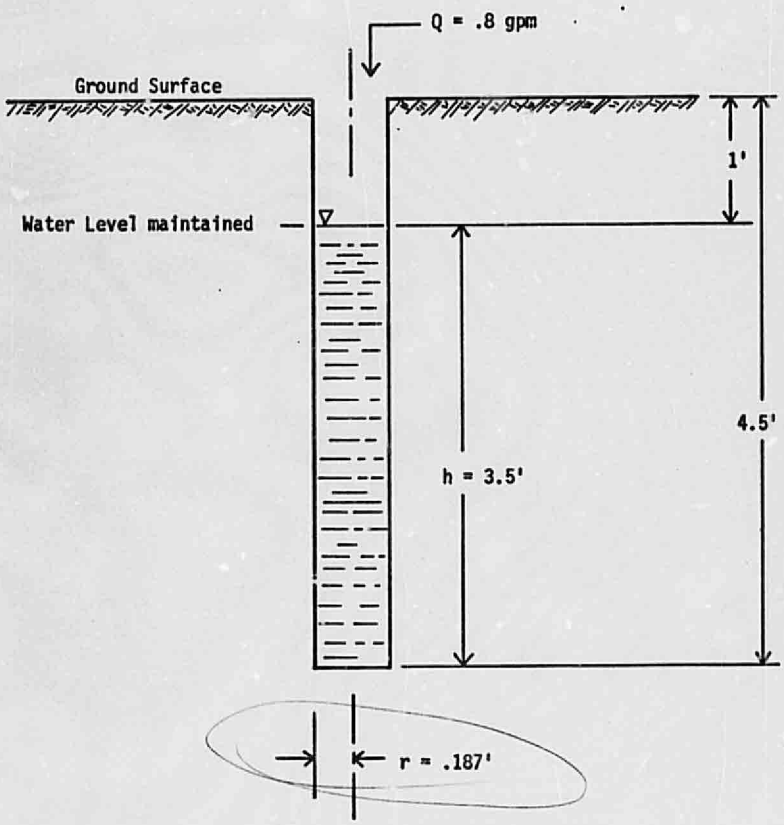


FIGURE 3 SKETCH SHOWING DETAILS

OF WELL PERMEAMETER TEST



Hydraulic Conductivity, $K = \dfrac{192\,[\sinh^{-1}(\frac{h}{r}) - 1]\,\frac{Q}{2\pi}}{h^2}$

$= \dfrac{192\,[\sinh^{-1}(\frac{3.5}{.187}) - 1]\,\frac{.8}{2\pi}}{3.5^2}$

$= 5.32$ ft/day $= 2.6$ in/hr.

The rational method of predicting a design peak runoff rate is expressed by the equation:

$$Q = C\ i\ A$$

where: Q = the peak runoff rate in cfs

C = the runoff coefficient

i = rainfall intensity in in./hr. for the design return period and for a duration equal to the "time of concentration" of the watershed

A = watershed area in acres

The time of concentration of a watershed is the time required for water to flow from the most remote (in time of flow) point of the area to the outlet. For the purpose of this study, time of concentration was estimated by the use of data presented in FIGURE 4 (SCS, 1973). Flow from northeast to southwest occurs as sheet flow and rill or gully flow are not evidenced. It appears that there is no inflow onto the site from uplying areas and all outflow from the site is produced by rainfall thereon.

Using FIGURE 4, a time of concentration velocity of 2.3 feet per second based on the computed slope of 5.45 percent is selected. If a flow distance of 275 feet is used, t_c = 2 min.

From NOAA Atlas 2, 1973, the six-hour-100 year rainfall is 2.6 inches in the vicinity of Thaxton and Girard, S.E. The point rainfall for a 24-hour-100 year storm is 3.0 inches.

Procedures for estimating the one hour precipitation are given in NOAA Atlas 2 for areas in New Mexico west of the Sangre de Cristo and Sacremento mountain ranges.

$$Y_{100} = .494 + .755\ [(X_3)\ (X_3/X_4)]$$

where:

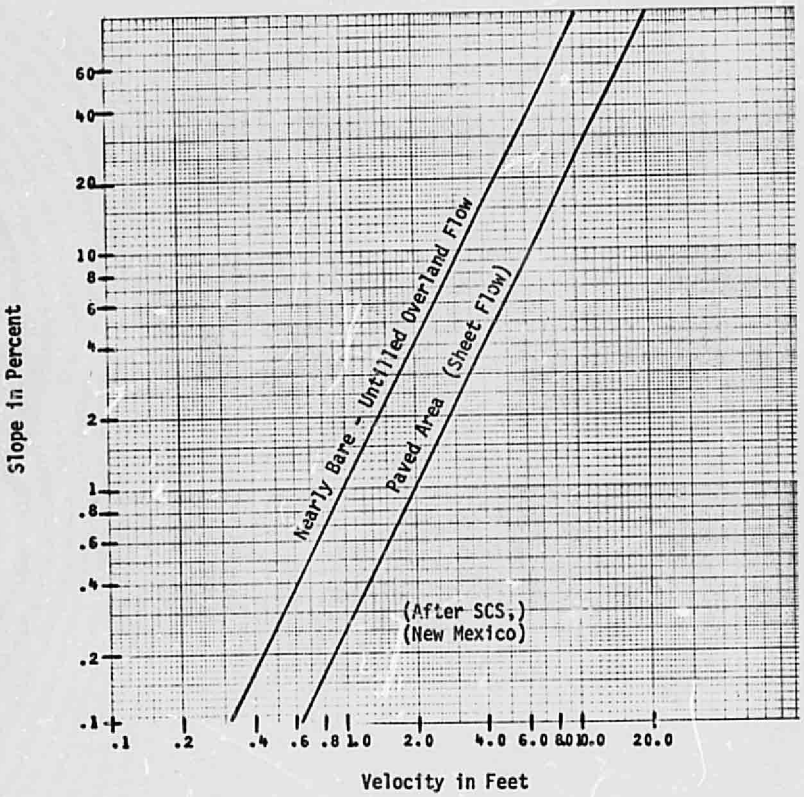
Y_{100} = 100 year-1 hour precipitation

X_3 = 100 year-6 hour precipitation

X_4 = 100 year-24 hour precipitation

FIGURE 4

VELOCITIES FOR UPLAND METHOD OF ESTIMATING "tc"



Then:

$$Y_{100} = .494 + .755\ [(2.6)\ (2.6/3.0)] \approx 2.2 \text{ in.}$$

The relationship on average one-hour rainfall intensities to intensities for shorter duration, but the same frequency, is presented in FIGURE 5. (Schwab, Frevert, Edminster and Barnes, 1966).

From FIGURE 5, $I_{60}/I_t = .12$

$$I_t = \frac{2.2}{.12} = 18.33 \text{ in./hr.}$$

The area of the site, which approximates 240 X 135 feet is .75 acres. The peak flow, Q , for a 100 year storm, then under present conditions would be estimated as:

$$\begin{aligned} Q_{100} &= C\ i\ A \\ &= (.35)\ (18.33)\ (.75) \\ &= 4.8 \text{ cfs} \end{aligned}$$

To estimate the total runoff volume (V_{100}) from the 100 yr.-6 hr. progress storm, the coefficient of runoff, C can be used as follows:

$$V_{100} = C\ d_{100}\ A$$

where:

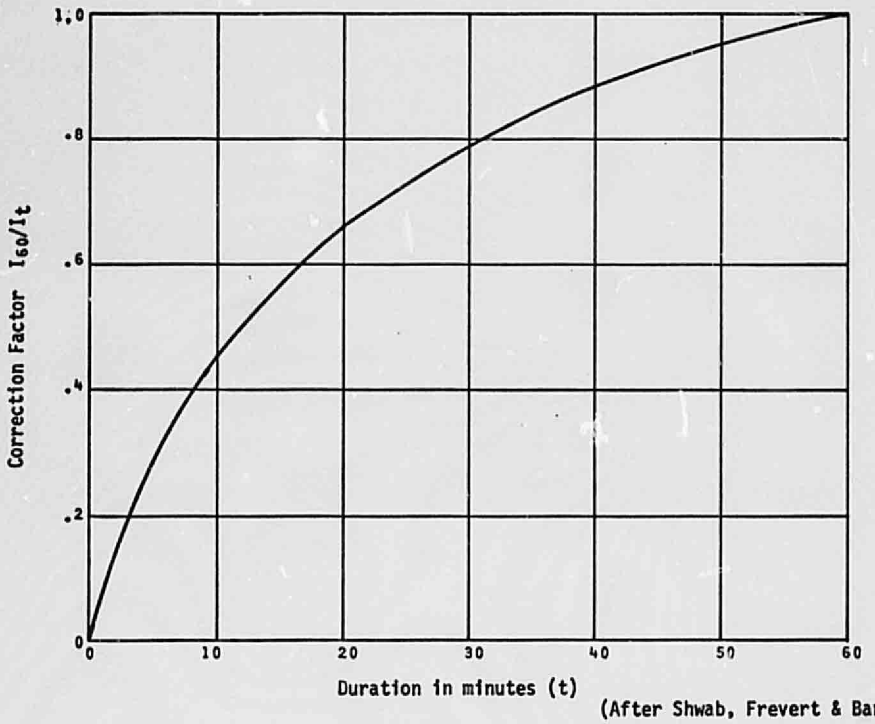
V_{100} = total storm runoff in acre-in.
d_{100} = 100 year-6 hr. point rainfall
A = area in acres

Runoff volume is then estimated for the site as:

$$\begin{aligned} V_{100} &= .35\ (2.6)\ .75 \\ &= .68 \text{ acre-in.} \\ &= 2{,}470 \text{ ft.}^3 \end{aligned}$$

All runoff from the site leaves the southwestern corner of the site and finds its way into a lined surface drainage channel flowing west above a storm sewer, some 230 feet south of the site. The water flows from this

FIGURE 5 RELATIONSHIP OF AVERAGE ONE-HOUR RAINFALL INTENSITIES TO INTENSITIES FOR PERIODS OF SHORTER DURATIONS



(After Shwab, Frevert & Barnes)

channel onto Ross Avenue, S.E., at Vassar, S.E., hence down Ross and into storm sewer inlets near Princeton and Ross. Two inlets to this same storm sewer are located on Girard, some 230 feet south of the site. In checking the sizing and grading of this storm sewer at the City Engineers office, it was found to be 27 inches in diameter between Girard and Vassar, increase to 30 inches in diameter for a short segment along Ross and reduce to 27 inches near Princeton. All three segments are on a grade of .032, indicating the sewer has the same capacity at Girard as it does near Ross and Princeton.

II. PROPOSED DEVELOPMENT AND DRAINAGE PLAN FORMULATION

The general site plan as envisioned by the architects and developers is shown in FIGURE 2. Basically, the site would be divided into five levels (benches) with the two-story, eight-plex units occupying levels 1, 3, and 5 and associated parking facilities would be located on levels 2 and 4. Creation of a high percentage of impervious surface on each level is quantified in TABLE I.

TABLE I

Impervious Areas By Level
(Areas in Square Feet)

Level	Roof	Sidewalk	Parking Lot	Total Impervious	Total Area	% Impervious
1	3,150	180	--	3,330	6,885	48
2	--	--	5,500	5,500	5,940	93
3	3,150	180	--	3,330	5,467	60
4	--	--	8,000	8,000	8,640	93
5	3,150	180	--	3,330	5,467	60

To assure that total runoff volume would not exceed that under existing conditions in the event of occurrence of the project storm and to further assure peak flow rates from the property would not be increased, the

following plan was formulated.

On levels 1, 3, and 5 where adequate impervious areas for ponding exist, an attempt would be made to retain and dissipate through infiltration all runoff generated on those levels.

On levels 2 and 4, where over 90 percent of the area will be impervious, runoff would be allowed but the time of concentration increased by using very low grades; thus, reducing peak flow rates generated.

For levels 1, 3, and 5, ponding areas will be established at both the east and west ends of each level. Areas of impondment are shown in TABLE II. Roof drainage would be such that one half would be discharged to the east and one half to the west. A "C" value of .95 was used for all impervious areas and of .30 for all pervious areas, assuming the establishment of cool season grass lawn. A "C" value of 1.0 was used for that precipitation falling directly on areas designated for ponding. Total inflow to each ponding area resulting from the project storm, resulting depth of ponding assuming no infiltration during inflow, required time for infiltration to occur and recommended average depth of ponding are shown in TABLE II. These levels will be graded so as to allow pervious areas to drain to the east.

TABLE II

Required Ponding Volumes and Infiltration Time

Level	Ponding Area	Area ($ft.^2$)	Inflow ($ft.^3$)	*Depth (in.)	Time for Infiltration (hr.)	Recommended Average Depth (in.)
1	East	632	655	12.4	4.8	8
1	West	500	432	10.4	4.0	6
3	East	629	569	10.8	4.1	6
3	West	405	412	12.2	4.7	8
5	East	629	569	10.8	4.1	6
5	West	405	412	12.2	4.7	8

* Assumes no infiltration during inflow

In establishment of time for infiltration, the intake rate was assumed equal to the hydraulic conductivity of 2.6 inches per hour established in the well permeameter test. Early in the period, intake rates would be higher but late in the period could be lower due to a higher inherent horizontal hydraulic conductivity than vertical in the soil strata.

An actual routing to set depths is not justified and recommended depths are conservative. Time of ponding would not be injurious to lawn grasses and grading requirements can be made architecturally attractive.

For Level 2 the impervious area with a runoff coefficient of .95 is 5,062 ft.^{2} and the pervious area with a runoff coefficient of .3 is 898 ft^2. The weighted average "C" value is .85. L is equal to 135 feet and the constructed slope to the east will be .001. From FIGURE 4, V = .6 ft./sec.; t_c then would be 3.75 min.

Level 4 would have the same "C" value and time of concentration.

From FIGURE 5 for a time of 3.75 min. the correction factor I_{60}/I_t = .23.

$$I_t = 2.2/.23 = 9.6 \text{ in./hr.}$$

For Level 2, Q = Cia = .85 (9.6) (.136) = 1.1 cfs

For Level 4, Q = Cia = .85 (9.6) (.198) = 1.6 cfs

Total peak flow from both parking areas would be 2.70 cfs. For the project storm a total runoff volume for the parking areas would be .738 acre inches or 2,678 ft^3.

III. JUSTIFICATION OF DRAINAGE PLAN

A. The plan provides full utilization of the pervious nature of soils at the site as ponding and infiltration areas allowing no runoff from Levels 1, 3, and 5.

B. Although drainage of the parking areas to Girard in the methods outlined increases total runoff volume from the site from the project storm by 8.4 percent, this is more than compensated for by the 43.8 percent reduction in peak flow rate from a 100 year storm.

C. Girard slopes at approximately 4.5 percent adjacent to the site to storm sewer inlets. Flow depths resulting from discharge on Girard would be negligible. Discharge would be into the same storm sewer as at present, but would enter upstream of the present flow pattern. Storm sewer capacity is constant through the reach.

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Schwab, Glenn O., R. K. Frevert, T. W. Edminister, and K. K. Barnes, 1966, Soil and Water Conservation Engineering, John Wiley and Sons, Inc., New York, N. Y.

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