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FILE

VILLAGE OF CLARENDON HILLS

VILLAGE-WIDE
STORM DRAINAGE STUDY

NOVEMBER, 1984

Prepared by:

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Introduction

With the continuing urbanization and redevelopment of the land in and around cities, the expansion of industry and the uninterrupted consumption of resources, our natural environment has undergone a serious deterioration in the last few decades.

For example, the construction of new housing and associated facilities such as schools, shopping centers, large parking areas and additional streets and driveways have altered the surface characteristics of watersheds so as to produce more runoff from rainfall or the melting of snow.

Because of this alteration in the earth's surface into more impervious conditions, the free water runoff no longer remains an integral part of the hydrologic cycle whereby precipitation infiltrates into the ground to be absorbed by the soil and vegetation or is temporarily stored in natural surface depressions and allowed to evaporate back into the atmosphere in continuation of the cycle. Instead, most of the runoff is quickly carried across sloping roofs, drives and lawn areas and is directed in relatively large volumes toward a combination of man-made and natural water collection facilities.

Consequently, the usage demands on the municipal collection and discharge systems have enlarged at a much greater rate than the systems themselves have physically expanded. Therefore, in many areas the culverts, storm sewers, open channels and other storm water collection structures are unable to generate sufficient capacity to discharge the storm flow thereby resulting in areas of flooding with the potential for property damage, loss of access and an exposure to dangerous, standing water conditions.

It is then in this perspective that the Village of Clarendon Hills has recognized the need for the identification of areas of existing or potential storm drainage deficiencies and for the evaluation of system modifications to minimize or eliminate these deficiencies.

Scope and Objective

As noted above, the intent or objective of an engineering study of this type is to produce meaningful data from which may be developed a realistic program of physical improvements in the storm drainage system to minimize flooding which is financially attainable by the Village.

To accomplish this overall objective, this report suggests the need for five separate phases of activity:

Phase 1: Problem Identification
Phase 2: Problem Solving (preliminary engineering designs)
Phase 3: Economic Considerations
Phase 4: Final Engineering (plans, specifications and estimates)
Phase 5: Construction

Each phase involves a series of sub-activities the combination of which produce a primary or phase objective. For example, Phase 1 introduces into the study seven sub-activities which, when executed, should satisfy this phase objective: the identification of drainage facilities now existing in the study area which are unable to discharge anticipated storm flows and thereby create hazardous conditions. These sub-activities are as follows:

A. Analysis of natural watershed configuration
B. Field reconnaissance to evaluate condition of existing drainage facilities
C. Establishment of realistic criteria which defines acceptable conditions of operation for storm drainage facilities
D. Hydrologic computations which develop quantities of rainfall expected to reach each drainage facility
E. Hydraulic calculations which develop quantities of rainfall each drainage facility is able to efficiently carry or pass
F. Comparison of each facility's hydraulic capacity with the quantity of flow generated by its watershed
G. Evaluation of individual facility conditions and development of improvement priorities

Phases 2 and 3 will analyze the hydraulic conditions for each facility requiring replacement or modification, will define the type of improvement required at each location and will establish estimates of construction cost so that the Village may effectively program the preparation of final plans and the construction of those structures or systems most in need of improvement at some time in the future.

Watersheds

As is shown on Exhibit 1, Existing Watersheds, in the appendix of this report, the Village is subdivided by fourteen major watersheds or independent drainage areas. The three watersheds generally located to the north of Chicago Avenue are tributary to Salt Creek and those south of Chicago Avenue are tributary to Flagg Creek.

Within each watershed there exists some natural storm water detention and/or retention areas which essentially consist of low points in the natural ground surface which hold water until the water rises to a certain elevation which allows it to flow out of the depression and continue its normal flow in the water course.

In addition to the natural detention/retention areas, there also exist five Village oriented, man-made detention/retention areas within the corporate limits of Clarendon Hills:

1. Hinsdale golf course lake and recently constructed detention area
2. Prospect Park Lake
3. Blue Lake
4. Park Avenue detention basin
5. Hudson Park detention basin

Existing Drainage Facilities

Within each watershed there exists a system or systems of storm drainage collection facilities which essentially may be subdivided into two categories:

1. Open ditch and culvert system where ditches or channels are either man made or natural (i.e. existing in nature) and the culverts are relatively short structures beneath roadways or other embankments and have either a rectangular or square cross section (box culverts) or a circular, elliptical or arch cross section (pipe culverts)

2. Storm sewer or closed drainage system where surface runoff is collected by inlets or catch basins located in the curb line of streets or other low areas and this flow is directed into a sewer which is usually of such length as to carry the storm sewage outside the urban area before discharge into channels or man-made ditches.

While both system types are encountered in Clarendon Hills, storm sewers or closed drainage systems tend to predominate.

In the hydrologic and hydraulic analyses of the existing drainage facilities, over 260 sewer segments were evaluated. The location, size and hydraulic evaluation data of the Clarendon Hills system has been summarized on Exhibit 2, Existing Storm Drainage Facilities, located in the appendix of this report.

Criteria

In the evaluation of each existing drainage structure's capacity to discharge storm flows, some criteria or means of judging must be established for both the anticipated intensity of rainfall (hydrology) and the hydraulic operation of the structure.

To determine the amount of storm runoff which is likely to reach any given structure, there are a number of formulae which produce reasonably accurate results such as the Rational Formula, the U.S. Soil Conservation method, the Burkli-Ziegler Formula and

Bureau of Public Roads equation. Because of its general acceptance by the engineering profession and its relatively simple application, the Rational Formula was used in this report to establish the quantity of flow generated by the watersheds.

This equation has the form Q = AiR where Q represents the quantity of flow in cubic feet per second, A is the watershed area in acres, i the rainfall intensity in inches per hour and R is the runoff coefficient. Individual watershed acreage was taken from contour interpretations as shown on Exhibit 1. The determination of rainfall intensities is somewhat more complex since rainfall for any geographical area varies from month to month and year to year. However, from U.S. Weather Bureau records, it is found that storms of certain intensities will be repeated within given intervals. The higher the intensity, the greater will be the time interval for recurrence. Therefore, if reference is made to a 10 year storm, it is suggested that this peak rainfall is expected to occur once in a 10 year period. Specific rainfall data for given times of storm frequency and concentration was obtained from Rainfall Intensity vs. Duration charts as developed by the Illinois Department of Transportation (IDOT) and appearing in the IDOT Design Manual. IDOT design policy and the generally accepted standard of the engineering profession establishes the use of a 10 year storm frequency for new storm sewer design.

In the analysis of the runoff generating characteristics of each watershed, three classifications of watershed development were established:

1. Residential: where approximately 45% of the rain which falls on the area is collected by the drainage system
2. Parks, playgrounds, golf courses: where 20% runoff generation is expected
3. Pavements, driveways, sidewalks, etc. Where 90% of the rain which falls on the area is collected by the drainage system.

Once the quantity of rainfall runoff that is expected to reach the drainage structure is developed, further criteria must be formulated from which to establish permissible limits of operation for the drainage facilities. These factors relate to allowing the hydraulic gradient (top of water surface) within the sewerage system to build up to the top of the appurtenant structures within that system such as manholes, catch basins or inlets but limiting the water surface to this elevation and not permitting discharge onto the adjacent properties.

Procedures and Hypotheses

1. Determine area upstream from system (Westmont)
2. Determine number of upstream detention areas, their combined tributary area, and their combined release rate
3. Determine Ave "R" value (runoff coefficient) for remainder of upstream areas, flow from 100% of area reaches Clarendon Hills (if there are pockets of low lying land that currently

do not have an outlet, as Westmont improves its storm sewer system, this water will eventually reach Clarendon Hills)

4. Any sewer not able to accept these storm flows will either cause ponding or over land flow
5. Because of cost restraints, wherever possible, ditches should be used in place of storm sewers
6. It is unlikely that Westmont will be willing to reduce the amount of the storm runoff to the east. This could be expensive and increase flooding problems in their town. If any additional stormwater detention ponds are to be constructed in Westmont, more than likely they would only contain the runoff from new development. Their release rate will approximate the current runoff of the undeveloped site. Thus, they will not be of much help in reducing the amount of stormwater coming to Clarendon Hills. (By the same reasoning, when analyzing upstream areas, assume undeveloped areas to remain undeveloped; do not assume them to be residential, etc. When (if) they are developed, the runoff/release rate won't change much because of the current stormwater retention requirements.)
7. When establishing priorities for work, one should attempt to enable the entire City system to accept a 2 year storm flow before considering 5 year or 10 year flows for certain areas. Projects receiving priority should return the most benefit per dollar.
8. When Clarendon Hills improvements will increase flow to the east (this will always happen unless we provide stormwater detention with restricted outflow) the entire receiving system should be analyzed to ensure that it can handle the increased flow. If it cannot, some sort of remedy will have to be coordinated with Hinsdale.
9. Due to the extensive nature of the storm sewer problems in Clarendon Hills, in addition to the fact that the existing system is so undersized, improvements are going to be costly and construction may have to be programmed over a number of years.

Existing System Evaluation, General Comments

Through the execution of the various activities and calculations described in preceding sections of this report, the quantity of flow generated by individual watersheds and the quantity of flow capable of being carried by the drainage structures can be computed. This information has been developed for storm frequencies of 2, 5 and 10 years and has been shown on Exhibit 2, Existing Storm Drainage Facilities.

The quantity of flow generated by the watershed in cubic feet per second is represented by the upper number or numerator of the fraction, and is predicated on the use of the Rational Formula. The quantity of flow that the drainage structure is capable of carrying is represented by the lower number or denominator of the fraction and is based on hydraulic computations which limit the operation of the structure so as to be in accordance with the factors listed in the Criteria phase of the report.

Obviously, when the fraction is less than 1, the structure is suitable hydraulically and will pass anticipated runoff for the storm frequency assigned. When the fraction exceeds 1, hydraulic ineffectiveness is indicated, with the degree of inefficiency noted by the amount by which unity is exceeded. Throughout the remainder of this report, the decimal equivalent of the ration described above shall be referred to as the Hydraulic Operation Number or HO Number.

Within the Village's system, 84 structure segments were analyzed hydrologically and hydraulically for storm recurrence frequencies of 2, 5 and 10 years and Hydraulic Operation Numbers calculated. The average HO Number for the 2 year storm was 4.8 with a range of 0.3 to 34.8; for the 5 year storm, the average HO Number was 6.0 with a range of from 0.5 to 42.9 and for the 10 year storm the average HO Number was 7.2 and ranged from a low of 0.7 to a high of 49.1.

Since the computations which form the basis for the H.O. Number are theoretical, it should not necessarily be assumed that every structure with an H.O. Number that exceeds 1 should be modified or replaced. Depending on specific circumstances, such as type of structure (culvert, sewer or inlet), location (open field, or residential area) and the actual quantities of flow involved; some facilities may be considered as usable even though the computations indicate a degree of hydraulic ineffectiveness. Each structure or system therefore, must be evaluated in the perspective of its individual circumstances and the framework of this section of the report's objective: the identification of existing facilities which cannot effectively discharge storm flows and create hazardous conditions.

EXISTING SYSTEM EVALUATION, SPECIFIC AREAS AND RECOMMENDATIONS FOR IMPROVEMENTS

The individual problem areas discussed herein are presented in the random order in which they were studied. No inference should be taken as to importance or priority of work from the order in which they are listed. As the cost of the projects vary widely, some may be lumped together; while others may be deferred until additional funding becomes available. Project priorities will be assigned based upon number of homes and other facilities (including streets) affected, severity of the problem, and cost. It should be noted that some projects depend on the completion of other projects to be effective.

A. PROSPECT PARK

The severity and frequency with which Prospect Park currently floods is a major problem. The long draw-down time of this area makes Oxford Avenue impassible and the homes along the west side of Oxford Avenue inaccessable for up to four to five days after a major storm. The present configuration of Prospect Lake does not provide sufficient stormwater detention volume to alleviate flooding in this natural (i.e. not man-made) basin. While of good size, the lake's normal water elevation is quite high. Oxford Avenue floods after the water rises just 20 inches.

It is recommended that Prospect Lake be expanded in size and reshaped to provide additional stormwater capacity. A pumping station can be used to lower the normal water level of the lake during "dry" periods (i.e. between rainstorms). The pump would discharge to the existing 12" outfall pipe and would be controlled by a float system. In designing the new lake configuration, emphasis should be placed on creating a facility that is aesthetically pleasing and that uses a minimum of park land currently dedicated to recreational activities. A preliminary plan has been prepared to show one possible lake configuration. An infinite number of shapes, sizes and depths may be used to accomplish the same stormwater detention volume. The total estimated cost of design and construction for this improvement is $144,300

B. JULIET COURT AT PROSPECT AVENUE

Severe flooding currently occurs along Prospect Avenue, between Norfolk Avenue and Chicago Avenue; and along Norfolk Avenue, between Prospect Avenue and Golf Avenue. However, the worst flooding in this area occurs at the intersection of Prospect Avenue and Juliet Court. The stormwater from this localized low area was once tributary to Prospect Park, but is now drained southward to Flagg Creek by an undersized (8") storm sewer.

It is proposed to construct roadside swales with driveway culverts along Prospect and Norfolk Avenues. A 36" storm sewer would then convey the runoff westerly, under Prospect Avenue, through the school district parcel, to Prospect Park. There it would be allowed to run overland, along the existing swale, to Prospect Lake. It should be noted that this project is proposed on the assumption that the Prospect Park project will be completed prior to construction. The total estimated cost of design and construction for this improvement is $46,400

C. NORFOLK AVENUE, EAST OF WOODSTOCK AVENUE

This area is a natural depression, with no positive outfall (i.e. homes will be flooded before stormwater reaches a depth sufficient to start draining the area by overland flow). This area is currently drained by an undersized (12" and 18") storm sewer system which travels west, and then south, to Blue Lake.

It is recommended that a storm sewer system beginning at 21" and increasing in size to 42", be installed along Norfolk Avenue to Woodstock Avenue; and then south to Blue Lake. One alternative to this routing is to construct a new storm sewer eastward along Norfolk Avenue to Prospect Park. However, based upon preliminary studies, it does not appear that this route will represent any cost savings. Again, if the route to Prospect Park is chosen, the Park improvement will have to be completed first. The total estimated cost of design and construction for this improvement is $111,500.

D. MIDDAUGH ROAD - RESTRICTORS

Some time ago, an earthen berm was constructed east of the practice area of the Hinsdale Golf Club. This was done to alleviate the flooding of homes on Middaugh Road, between Walnut and Maple Streets. Since that time, it has been observed that when the detention area behind the berm fills up with water, downstream inlets, mainly those near the golf course's east property line, form "geysers". The stormwater coming out of these inlets then floods the rear yards of the homes along Middaugh Road.

It is proposed that the inlet which drains the water from the practice area be modified to restrict its intake capacity. This will reduce the pressure on downstream pipes and prevent surcharging. Restricting the inlet will result in the detention area filling more rapidly and to a greater degree during storms. It should be noted that while this modification will have a beneficial effect during "small" storms (2 year, 5 year, and 10 year storms) it will have no effect on "larger" storms (25 year, 50 year, and 100 year storms). This is because the detention area will then fill up and overflow to the east, where the runoff will flood the rear yards. The total estimated cost of design and construction for this improvement is $600.

E. MIDDAUGH ROAD - BERM

Even with the restriction of the inlet in the golf course practice area, the homes on Middaugh Road, between Walnut and Maple Streets will continue to experience flooding problems during severe storms. The existing storm sewer system (twin 18" pipe) is inadequate to handle the stormwater runoff.

The construction of a large storm sewer from the golf course to Route 83 is impractical due to cost. It is recommended that a berm be constructed in the rear yards of the affected homes. Stormwater could thus be stored until the existing storm sewer could transport it downstream. The total estimated cost of design and construction for this improvement is $25,600.

F. HARRIS AVENUE, FROM WALKER AVENUE TO EASTERN AVENUE

There is currently a low area approximately 200 feet south of, and parallel to, Harris Avenue. The inlets on Harris Avenue are too high to drain this area and overland flow to Flagg Creek is prevented by the elevation of downstream residential development. Even if the runoff from this area could reach the existing storm sewer system, it is undersized and could not handle a major storm. (The existing storm sewer is 24" in diameter).

It is proposed that a parallel storm sewer system be constructed along Harris Avenue, with inlet branches extending southerly along Grant Avenue and Prospect Avenue. This storm sewer would vary in size from 10" at Walker Avenue to 42" at Holmes Avenue. The total estimated cost of design and construction for this improvement is $114,900.

G. 55TH STREET, AT WALKER AVENUE

Because of residential development along 55th Street and Ruby Street, the natural drainage path to the low area in Hudson Park has been blocked. The area is currently drained by an inadequate (12") storm sewer to Flagg Creek.

It is suggested that an additional 30" and 36" storm sewer be constructed north along Walker Avenue to Hudson Street; and then west to Hudson Park, to the existing stormwater detention basin. This project would relieve flooding at 55th Street, would return the runoff to its proper watershed and would lessen a storm's impact on the area further north on Harris Avenue. The total estimated cost of design and construction for this improvement is $72,500.

H. TRAUBE AVENUE AT WOODSTOCK AVENUE

This area is a natural depression with no positive outfall. It has a large (183 acre) tributary area which extends into Westmont; and experiences severe flooding during even moderate storms. It is currently drained by a 24" storm sewer which is not adequate to prevent flooding.

It is recommended that additional inlets be constructed at the intersection and that these be connected to a proposed 60" storm sewer which would run along Traube Avenue from Woodstock Avenue to Oxford Avenue. At Oxford Avenue, a ditch would be constructed to the north. At the bottom of the hill, the runoff would drain easterly to a low point on the golf course. This low point, with its 24" outlet pipe currently acts as a natural stormwater detention pond. It is predicted that the effect of this project on the golf course operation will be minimal. The total estimated cost of design and construction for this improvement is $157,500.

I. CHESTNUT AVENUE

There is currently a problem with flooding at the west end of Chestnut Avenue. This area drained to the ditch that lies to the north of the Burlington Northern Railroad tracks at one time. However, development south of Chestnut Avenue has blocked the natural drainage. The area is now serviced by a 10" storm sewer which is undersized.

It is proposed to construct a 15" storm sewer from the existing inlets east of Golf Avenue, west and south, to the Burlington Northern ditch. The total estimated cost of design and construction for this improvement is $21,000.

J. BURLINGTON AVENUE

Burlington Avenue, west of Hiawatha Drive and near the west limits of Clarendon Hills, is the discharge point for a large drainage basin, most of which lies in Westmont. This drainage basin encompasses 108 acres and the storm runoff is discharged by Westmont through a 30" and 42" storm sewer. Clarendon Hills attempts to accept this water with a single 36" x 58" pipe; which is reduced downstream to a single 42" storm sewer. The excess water is forced out through the inlets in the street and floods all low-lying areas on its overland journey to Blue Lake.

It is suggested that an additional 36" storm sewer be constructed from the inlets west on Iroquois Drive to the inlets west of Mohawk Drive; and from this point, a 60" storm sewer be constructed to the manholes east of Indian Drive. From this point, a 40' wide swale can be constructed between Burlington Avenue and the railroad tracks which will convey the stormwater to Blue Lake. The total estimated cost for design and construction of this improvement is $111,500.

K. RUBY STREET, EAST OF RICHMOND AVENUE

Approximately 250 feet east of Richmond Avenue, Ruby Street has a low spot. The houses on the south side of Ruby Street are quite a bit higher than the homes on the north side and drainage is northerly. There is an existing inlet in the north parkway which is connected to a private 6" storm line that runs north between two houses. The front yards of homes on the north side of the street are nearly flat and have great potential for flooding during heavy rains. The inlets on Richmond Avenue are too high to be of any help.

Because the existing storm sewer on Richmond Avenue is too high, it is proposed to relay 350 feet of 15" storm sewer to a greater depth between Ruby Street and Hudson Avenue. A 15" pipe can then be laid eastward to the existing inlet. Additionally, a swale should be constructed along the north parkway to ensure that water in the street is routed to the inlet and not northward toward homes. The total estimated cost of design and construction for this improvement is $20,000.

JAMES J. BENES AND ASSOCIATES, INC.
1100 JORIE BLVD., OAK BROOK, ILLINOIS 60521
TELEPHONE: (312) 654-4344

August 16,1985

Village of Clarendon Hills
One North Prospect Hills
Clarendon Hills, Illinois 60514

Attn: Ed Glatfelter
Village Manager

Re: Drainage Problem
215 Coe Road
Project No. 228

Dear Mr. Glatfelter:

This letter is in response to your request that this office conduct a study of the flooding problem along Maple Street, from Coe Road to North Jackson Street. When we prepared our village-wide storm drainage study last November, this area was not addressed. In that study it was not our intention to solve every flooding problem in Clarendon Hills, but to relieve flooding only in those areas that were most severe. If we had been aware of the drainage problem at Maple Street and Coe Road last autumn, we would have included it in the village-wide study.

The drainage basin tributary to this area is approximately 12 acres in size and extends southerly, to Chicago Avenue; westerly to Middaugh Road; and northerly, 150 feet north of Maple Street. At one time, this area drained through a natural channel to the east. The house at 215 Coe Road is located in this low area and is currently experiencing flooding problems in both front (west) and back (east) yards.

The storm sewer in this area consists of a 12" culvert across Coe Road, with an inlet on the east side of Coe Road which is connected to an inlet manhole at the southeast corner of Coe Road and Maple Street via a 15" storm sewer. A 15" culvert crosses Maple Street into this inlet manhole and a 15" storm sewer extends easterly from the manhole approximately 200 feet. From there, ditch flow carries the runoff to a two foot square box culvert under Route 83, with 15" culverts under two driveways at 45 Maple Street and twin 29"x18" culverts under North Jackson Street. The culvert under Coe Road is undersized, as are the 15" storm sewer and driveway culverts at 45 Maple Street. The twin 29"x18" culverts under North Jackson Street are adequate.

In order to relieve the flooding in front of 215 Coe Road it is recommended that an 18" storm sewer be constructed parallel to the existing 15" storm sewer. In order to relieve the flooding in the area between 215 Coe Road and 45 Maple Street, it is suggested that the existing 15" driveway culverts at 45 Maple Street be replaced with 21" culverts; and that the ditchline along the front of the lot be regraded to allow positive drainage and sufficient ditch capacity. I have included a profile of the existing ditchline on Maple Street. It shows that a portion of this ditch is backpitched and currently does not drain. The total estimated cost for design and construction of this improvement is $ 12,600.

If you have any questions or comments, please feel free to contact me or Jim Benes.

Very truly yours,

JAMES J. BENES AND ASSOCIATES, INC.

James E. Darnell

by: James E. Darnell, P.E.

PRELIMINARY ENGINEER'S ESTIMATE OF COST

	DESCRIPTION	ESTIMATED CONST. COST	ESTIMATED DESIGN COST	ESTIMATED CONST. INSP. COST	TOTAL EST COST
A.	Prospect Park	$ 160,000	$ 9,600	$ 8,000	$ 177,600
B.	Juliet & Jane Ct.	41,000	2,900	2,500	46,400
C.	Norfolk & Woodstock	100,000	6,000	5,500	111,500
D.	Restrictors on Golf Course Inlet	600	--	--	600
E.	Middaugh Berm	22,000	1,800	1,800	25,600
F.	Harris Avenue Walker to Eastern	103,000	6,200	5,700	114,900
G.	Walker Avenue 55th St. to Hudson	64,000	4,500	4,000	72,500
H.	Woodstock & Traube	145,000	6,500	6,000	157,500
I.	Chestnut Avenue	18,000	1,500	1,500	21,000
J.	Burlington Avenue	100,000	6,000	5,500	111,500
K.	Ruby & Richmond	17,000	1,500	1,500	20,000
L.	Maple & Coe	11,000	900	700	12,600
		$781,600	$47,400	$42,700	$871,700