FINAL DRAINAGE REPORT
FOR
CASA GRANDE P.U.D.
FORT COLLINS, COLORADO

PREPARED FOR:
PROGRESSIVE LIVING STRUCTURES, INC.
MAY 1985

PREPARED BY:
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Revised: July, 1985
Introduction: Casa Grande P.U.D. is located in the Southwest corner of Section 27, T7N, R69W of the 6th PM in the city of Fort Collins, Colorado. The property was originally a part of the third filing of Wagon Wheel Subdivision and lies in the Foothills Drainage Basin. This report will analyze the effects, on the proposed condominium development, of a 2 year and 100 year storm event. The drainage plan will closely follow the master drainage plan for the Wagon Wheel Subdivision by Melody Homes.

Summary: Casa Grande P.U.D. will utilize 2 detention facilities in Basins III and IV. The total detention storage required for these two basins is 0.66 Acre-Feet. The release from each detention facility is based on the 2 year developed runoff which is directed to the storm sewer system in Laredo Lane. The runoff from the remaining areas defined as Basins I, II, and V will be handled by the adjacent streets and eventually be directed to Tract 4.

The calculated historic runoff for a 100 year frequency storm across the site, not including offsite flows, is 8.3 cfs. The 100 year flows generated from Basins I, II and V is slightly higher than the historic 8.3 cfs but includes portions of Seneca and Hickok, west of, and not considered a part of this development. If the street contributions from west of Casa Grande for Basins I, II and V were deleted, the 100 year developed flow to reach Tract 4 would be \((0.50)(4.6)(2.74)(1.25)=7.9\text{cfs} \approx 8.3\text{cfs}\). The 100 year flow to reach Tract 4, including street contributions from west of Casa Grande would be approximately \(0.60(4.6)(3.11)(1.25)=10.7\text{cfs} > 8.3\text{cfs}\).


Final Drainage Study, Wagon Wheel Subdivision Filings Number 1, 2 and 3. Final Revision Date 5/20/81, Melody Homes.
<table>
<thead>
<tr>
<th>Location of Design Point</th>
<th>Basins</th>
<th>Length ft.</th>
<th>Inlet Time min.</th>
<th>Street min.</th>
<th>Pipe min.</th>
<th>Time of Concentration min.</th>
<th>Coefficient C</th>
<th>Intensity in. / hr.</th>
<th>Area in.² / acre</th>
<th>Direct Roof Runoff</th>
<th>Other Roof Runoff</th>
<th>Sediment Runoff Cts</th>
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FIGURE 5-1. TYPICAL FORM FOR STORM DRAINAGE SYSTEM PRELIMINARY DESIGN DATA
Historic runoff from property

Total area = 10 acres

Largest travel distance = 930'
\[ \Delta \text{elev.} = 92 - 80 = 12' \]
Avg. Slope = 1.29%

\[
T_{c1} = \frac{1.87[1.10 - 0.20(1.0)(1.0)](930)^{1/2}}{(1.29)^{1/3}} \]

\[
T_{c2} = \frac{1.87[1.10 - 0.20(1.0)7(930)^{1/2}}{(1.29)^{1/3}} = 47 \text{ minutes} \quad i_2 = 1.10 \text{ in/hr.}
\]

\[
T_{c100} = \frac{1.87[1.10 - 0.20(1.25)](930)^{1/2}}{(1.29)^{1/3}} = 44.5 \text{ minutes} \quad i_{100} = 3.30 \text{ in/hr.}
\]

Q_2 = 0.20(1.10)(10.0)(1.0) = 2.2 cf/s

Q_{100} = 0.20(3.3)(10.0)(1.25) = 8.3 cf/s
Basin I to Concentration Point A

Pervious area = 0.80 acres
Impervious area = 1.02 acres
Total area = 1.82 acres

Conv. = \frac{0.85(1.02) + 0.80(0.80)}{1.82} = 0.62

\text{Longest travel distance} = 1260'
\Delta \text{elev.} = 910 - 824 = 86'
\text{Avg. slope} = 0.68\%

Tc = \frac{1.87[(1.10 - 0.62)(1.10)]^{1/2}}{5^{1/3}}

Tc = \frac{1.87[(1.10 - 0.62)(1.10)]^{1/2}}{(68)^{1/3}} = 36 \text{ minutes}
\text{c} = 1.3 \text{ in/ hr.}

T_{100} = \frac{1.87[(1.10 - 0.62)(1.25)]^{1/2}}{(68)^{1/3}} = 24.5 \text{ minutes}
\text{c}_{100} = 4.6 \text{ in/ hr.}

Q_{c} = 0.62(1.3)(1.82)(1.0) = 1.5 \text{ cfs} < 8.9 \text{ cfs} = \text{street capacity (No inlet)}

Q_{100} = 0.62(4.6)(1.82)(1.25) = 65 \text{ cfs} < 8.3 \text{ cfs (100 yr. historic)}}
**Basin II to Concentration Point B**

**Pervious area = 0.24 acres**

**Impervious area = 0.34 acres**

**Total area = 0.58 acres**

\[
C_{av} = \frac{0.95(0.24) + 0.20(0.34)}{0.58} = 0.64
\]

**Longest travel distance = 290’**

**Δ e elev = 87.0 - 85.16 = 1.84**

**Avg. slope = 0.60%**

\[
T_c = \frac{1.87[1.10 - C_{av}]}{S^{1/3}}
\]

\[
T_{c2} = \frac{1.87[1.10 - 0.64(1.0)] 290^{1/2}}{(0.60)^{1/3}} = 17.4 \text{ minutes} \quad i_2 = 2.0 \text{ in/hr}
\]

\[
T_{c10} = \frac{1.87[1.10 - 0.64(1.25)] (290)^{1/2}}{(0.60)^{1/3}} = 11.3 \text{ minutes} \quad i_{10} = 6.90 \text{ in/hr}
\]

\[
Q_c = 0.64(2.0)(0.58)(1.0) = 0.7 \text{ cfs}
\]

\[
Q_{10} = 0.64(6.90)(0.58)(1.25) = 3.2 \text{ cfs}
\]
Basin IV to concentration point C - Detention Pond #1

Pervious Area = 0.95 Acres
Impervious Area = 1.18 Acres

Total Area = 2.13 acres

\[
\text{Cavg.} = \frac{0.95(1.10) + 0.95(0.95)}{2.13} = 0.62
\]

Longest travel distance = 360'
\( \Delta \text{elev.} = 86.5 - 81.0 = 5.50' \)
Avg. Slope = 1.45%

\[
T_o = \frac{1.87[1.10 - CC_f](D^{1/3})}{S^{1/3}}
\]

\[
T_o = \frac{1.87[1.10 - 0.62(1.0)](380)^{1/2}}{(145)^{1/3}} = 15.5 \text{ minutes} \quad \frac{i_2}{2.1 \text{ in/hr.}}
\]

\[
T_{100} = \frac{1.87[1.10 - 0.62(1.25)](380)^{1/2}}{(145)^{1/3}} = 10.5 \text{ minutes} \quad \frac{i_{100}}{7.0 \text{ in/hr.}}
\]

\[
Q_o = 0.62(2.1)(2.13)(1.0) = 2.8 \text{ cfs}
\]

\[
Q_{100} = 0.62(7.0)(2.13)(1.25) = 11.5 \text{ cfs}
\]

Required detention = 0.17 acre ft. (See mass diagram for det. pond #1)

Detention pond release rate = 2.8 cfs (Use 12" RCP @ 0.61%)
Basin IV to Concentration Point D - Detention Pond #2

Pervious Area = 2.19 acres \( (C=0.20) \)
Impervious Area = 2.94 acres \( (C=0.95) \)
Total Area = 5.13 acres

\[
C_{avg} = \frac{0.95(2.94)+0.20(2.19)}{5.13} = 0.63
\]

Longest travel distance = 943'
\[
\Delta \text{elev.} = 91 - 79.20 = 11.80
\]
Avg. Slope = 1.25 %

\[
T_c = 1.87 \left[ \frac{1.10-C_{avg}}{D^{1/3}} \right] \]

\[
T_c = 1.87 \left[ \frac{1.10 - 0.63}{(943)^{1/3}} \right] = \text{25 minutes} \quad i_2 = 162 \text{ in/hr}
\]

\[
T_{c100} = 1.87 \left[ \frac{1.10 - 0.63(1.25)}{(1.25)^{1/3}} \right] = \text{16.7 minutes} \quad i_{100} = 56 \text{ in/hr}
\]

\[
Q_c = 0.63(162)(5.13)(10) = 52 \text{ cfs}
\]

\[
Q_{100} = 0.63(56)(5.13)(1.25) = 22.6 \text{ cfs}
\]

Required detention = 0.49 acre ft. (See mass diagram for detention pond #2)
Detention pond release rate = 5.2 cfs (Use 15" RCP @ 0.65%)
Detention Storage Determination

Basin III - Detention Pond #2

(Mass Diagram)

Cumulative Runoff - 100 year frequency (gpm)

Required Storage Volume = 0.49 Acre ft.

Discharge to Storm Sewer System: Qd = 52 cfs

7/1/85
Project No. 1005-1
Basin III and Basin IV to Concentration Point D-1; Storm manhole

(to size storm sewer)

Cavg. = \( \frac{0.62(2.13) + 0.63(5.13)}{726} \) = 0.63

Tc = 15.5 minutes \( i_e = 21\ \text{"/hr.} \) (See Basin III calculations)

Qc = 0.63(2.1)(726)(110) = 9.6 cfs

8.15 cfs (from west of Casa Grande)

Total storm sewer capacity: 17.75 cfs

21" RCP @ 1.25%

or

24" RCP @ 0.61%
Basin II and Basin VII to Concentration Pt. E, to size inlet

Area of Basin II = 0.58 acres  
Area of Basin VII = 4.35 acres  
Total Area = 4.93 acres  

C = 0.66 (worst case condition from Basins I & VII Calculations)

Longest travel distance = 1085'  
Δ elev. = 870 - 7970 = 730'  
Avg. Slope = 0.67%

\[ T_e = \frac{1.87[1.10 - 0.66(1.1)](1085)^{0.6}}{(0.67)^{1/3}} = 31 \text{ minutes} \]

\[ Q_e = 0.66(143)(4.93)(1.0) = 47 \text{ cfs} \]

A city of Ft Collins std 4' curb inlet is not adequate (see figure 5-2)  
Use a 5' type R inlet.

Storm sewer lateral required; 15°RCP @ 0.53% or greater.
For basin III

\[ \text{Pervious area} = 0.05 \text{ acres} \]
\[ \text{Impervious area} = 0.30 \text{ acres} \]
\[ \text{total area} = 0.35 \text{ acres} \]

\[ C_{av} = \frac{0.95(0.30) + 0.20(0.05)}{0.35} = 0.84 \]

For basin I and III

\[ C_{av} = \frac{0.84(0.35) + 0.62(1.82)}{2.17} = 0.66 \]

\[ \text{Largest travel distance} = 1570' \]
\[ \Delta \text{elev.} = 940' - 797' = 113' \]
\[ \text{Avg. Slope} = 0.72\% \]

\[ T_{c2} = \frac{1.87[110 - 0.66(10)](1570)^{1/2}}{(0.72)^{1/2}} = 36 \text{ minutes} \]
\[ i_2 = 1.3 \text{ in/hr} \]

\[ Q_c = 0.66(1.30)(2.17)(10) = 19 \text{ cfs} \]

City of Ft Collins 4' curb inlet is adequate (see figure 5-2)
Basins I, II, III, IV, VI and VII to Concentration Point G

(to size storm sewer)

Total Area = 14.36 acres

C = 0.66 (worst case condition from Basins I and VII calculations)

Tc = 31 minutes (worst case condition from Basins II & III calculations)

i_c = 1.43

Q_c = 0.66 (1.43) (14.36) (10) = 13.55 cfs

8.15 cfs from west of Casa Grande

Total = 21.70 cfs

Use 24" RCP @ 0.92% or greater.
Street Capacity Analysis for Laredo Lane, (South Half), From Section 4 - Streets of the City of Fort Collins, Storm Drainage Criteria Manual.

\[ Q = 0.56 \left( \frac{\varnothing}{n} \right) S^{1.8/3} \]

For Area b
\[ y = 0.34' \]
\[ n = 0.016 \]
\[ s = 0.00605 \]
\[ g = 1 ''/02 = 50 \]

\[ Q_b = 0.56 \left( \frac{50/0.016}{0.00605} \right)^{\frac{1}{2}} \left(0.34\right)^{8/3} \]
\[ Q_b = 7.7 \text{ cfs} \]

For Area a
\[ y = 0.45' \]
\[ n = 0.016 \]
\[ s = 0.00605 \]
\[ 1/\varnothing = \left( \frac{1}{11/14/12} \right) = 0.94 \]
\[ \varnothing = 10.60 \]

\[ Q_a = 0.56 \left( \frac{10.60/0.016}{0.00605} \right)^{\frac{1}{2}} \left(0.45\right)^{8/3} \]
\[ Q_a = 3.4 \text{ cfs} \]

Q Total = 7.7 cfs + 3.4 cfs = 11.1 cfs

Theoretical Capacity for Laredo Lane (South 1/2 Street) = 11.1 cfs.

Apply reduction factor from figure 4 - 2
\[ F = 0.80 \]
Allowable street capacity = 0.80 (11.1) = 8.9 cfs
Street Capacity Analysis for Laredo Lane, (North Half), From Section 4 - Streets of the City of Fort Collins Storm Drainage Criteria Manual.

\[ Q = 0.56 \ (\frac{h}{n}) \frac{s^{y/3}}{14' \ \text{Pavement} \ (16.83')} \]

For Area \( b \)

\[ y = 0.34' \]
\[ n = 0.016 \]
\[ s = 0.00657 \]
\[ z = 1/0.02 = 50 \]
\[ Q_b = 0.56 \ (50/0.016) (0.00657)^{1/5} (0.34)^{8/3} \]
\[ Q_b = 8.0 \text{ cfs} \]

For Area \( a \)

\[ y = 0.45' \]
\[ n = 0.016 \]
\[ s = 0.00657 \]
\[ 1/z = (0.11/(14/12)) = 0.094 \]
\[ z = 10.6 \]
\[ Q_a = 0.56 \ (10.60/0.016) (0.00657)^{1/5} (0.45)^{8/3} \]
\[ Q_a = 3.6 \text{ cfs} \]

\[ Q \text{ Total} = 8.0 \text{ cfs} + 3.6 \text{ cfs} = 11.6 \text{ cfs} \]

Theoretical Capacity for Laredo Lane (North 1/2 Street) = 11.6 cfs.

Apply reduction factor from figure 4 - 2
\[ F = 0.80 \]
Allowable street capacity = \[ 0.80 \ (11.6) = 9.3 \text{ cfs} \]
Private Driveway Capacity Analysis from Section 4 - Streets of the City of Fort Collins, Storm Drainage Criteria Manual.

\[ Q = 0.56 \left( \frac{A}{n} \right)^{\frac{1}{2}} y^{8/3} \]

For Basin III

\[
\begin{align*}
&y = 0.24' \\
n & = 0.016 \\
s & = 0.005 \\
& z = T/4 = 24/0.24 = 100 \\
\end{align*}
\]

\[ Q = 0.56 \left( \frac{100}{0.016} \right) (0.005)^{\frac{1}{2}} (0.24)^{8/3} \]

\[ Q = 5.5 \text{ cfs} \text{ (for Channel Slope of 0.50\%)} \]

Apply reduction factor from figure 4 - 2

\[ F = 0.80 \]

Allowable private drive capacity = 0.80 (5.50) = 4.4 cfs
Figure 4-1
NONOGRAPH FOR FLOW IN TRIANGULAR GUTTERS
(From U.S. Dept. of Commerce, Bureau of Public Roads, 1965)
Figure 5-2
NOMOGRAPH FOR CAPACITY OF CURB OPENING INLETS IN Sumps, DEPRESSION DEPTH 2"
Adapted from Bureau of Public Roads Nomograph

MAY 1984
5-10
DESIGN CRITERIA