HYDROLOGY STUDY

FOR

Thrifty Oil – Perris Industrial

IN THE

COUNTY OF RIVERSIDE, CALIFORNIA

February 17, 2023

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TABLE OF CONTENTS

INTRODUCTION

The purpose of this drainage analysis is to ensure that the proposed drainage system is cable to safely convey and discharge Q_{100} flows in a manner that perpetuates the existing, undeveloped conditions. Additionally, the project's grading shall be designed to meet Riverside County Flood Control & Water Conservation District (R.C.F.C. & W.C.D.) criteria, guidelines and the Master Drainage Plan (M.D.P.) for Perris Valley.

Thrifty Oil currently owns project parcels APN 317-260-016 (8.70 AC) and parcel (APN 317-260-015) (0.44 AC). This project entails grading of the existing site for the development of an industrial facility at the northeast corner of the Tobacco Road and Water Street intersection. The site consists of 9.1 acres and is currently undeveloped and barren land consisting of sparsely spread weeds and grasses. The surrounding areas all have similar conditions.

Per the MDP, the 40-acre site bounded by Water St., Harvill Avenue, Tobacco Road and Placentia Ave., is tabled to drain northeast to an existing 66-inch storm drain at Harvill/Placentia intersection, after the confluence of Lateral H 10 (in Harvill) and Lateral H 10.1 (in Placentia). However, based on the existing topography, our project site actually drains easterly across the APN 317-260-034 (County of Riverside owned undeveloped parcel), via sheet flow at the common property line boundary (easterly p.l. Thrifty, westerly p.l. County). ***Per drainage case law, our natural drainage condition is permitted to perpetuate existing conditions as long as the flow is not concentrated, and the volume and velocities emitted from the site are less than the predeveloped conditions.*** RCFD & WCD has indicated that interim detention facilities are not required if the 100-year flow-rate is conveyed to the 66-inch RCP outlet at Harvill/Placentia. However, to accomplish such would require a new parallel storm drain in Placentia to Harvill as there is no capacity in Lateral H 10.1 or a new storm drain across the County parcel in an easement. Neither alternative is desirable.

A hydraulic analysis was performed to determine if there was any available capacity in Lateral H 10.1 (Placentia) to accept project flows. Calculations support the conclusion that Lateral H-10.1 does not have sufficient capacity to accept post-developed Q_{100} flows from the site (nor any other project in the area). Due to the lack of adequate capacity in Placentia and the desire to not encumber the County parcel with a storm drain and easement, flows from the site will be routed to an underground detention/infiltration system where the design capture volume (DCV) of runoff will be treated through infiltration. All runoff that exceeds the site's DCV will be discharged from the underground facility via overflow pipes. These overflow pipes will discharge into a concrete U-shaped channel constructed with side weirs where flows will be dispersed at a rate per linear foot less than or equal to pre-developed conditions. A grouted rip-rap stabilization blanket will be provided along our project easterly border, to protect the County parcel and help disperse the minimal flow Q-100 discharge of only 0.019 c.f.s. per linear foot.

The site is complying with the Riverside County Flood Control & Water Conservation District's interim detention basin facilities requirements since there is no practical storm drain outlet provided for the parcel per the current M.D.P. We are solving that issue by perpetuating the natural drainage conditions, while providing an effective safe and practical point of discharge. We are providing underground storm drain detention basin systems in similar concept and principal to recently approved project located to our northwest along Patterson and Placentia Avenues per PPT 190008 "Barker Logistics", with a safe weir dispersal structure, minimal flows for the Q-100, a free draining facility chamber basin within 72-hrs., and no dead storage or pumping facilities. Infiltration techniques are also provided to meet regional WQMP best management practices, and enhance water quality and quantities in the drought stricken southern California climate.

DESIGN CRITERIA

This Drainage Study was based on the following Design Criteria identified by the City of Perris:

1. Tributary flows must be routed through the site in a safe and non-destructive manner. The storms studied include the 1-hour, 3-hour, 6-hour, and 24-hour, duration events for the 2-year, 5-year, and 10-year return frequencies. The underground detention chamber will ensure none of these storm events have a higher peak discharge in the post-developed condition than in the pre-developed condition.

2. The underground detention chamber will be capable of passing the 100-year storm without damage to the facility.

3. Flows discharged off site will be less than or equal to pre-developed flows at a rate per linear foot.

4. Basins will drain in 72 hours or less.

DESIGN METHODOLOGY

1. **RATIONAL METHOD** – The Rational Method was used to determine the peak discharge for watersheds less than 1 square mile. Civil Design Software by Bonadiman and Associates was used to model the RCFC & WCD rational method for the project site. The following design data was used within our model:

    A. Rainfall (Rational) – Plate D4.1 of Riverside County Hydrology Manual was used to provide duration vs. intensity data for Perris.

    B. Infiltration – Soil Type "A" underlies the site area based on a review of the hydrologic soils group map for Perris.

    C. Antecedent Moisture Content (AMC) –AMC II was used for a for the 10-year event and AMC III was used for the 100-year event based on Riverside County Hydrology Manual's recommendations for use with the above storms.

2. **UNIT HYDROGRAPH** – Unit hydrographs were used to determine the peak volume for the watersheds. Civil Design Software by Bonadiman and Associates was used to model the RCFC & WCD unit hydrographs for the project site. The following design data was used within our model:

    A. Rainfall (Rational) – Plate D4.1 of Riverside County Hydrology Manual was used to provide duration vs. intensity data for Perris.

    B. Antecedent Moisture Content (AMC) –AMC I was used for a Unit Hydrograph for the 2 and 5-year, AMC II was used for a Unit Hydrograph for the 10-year, and AMC III was used for a Unit Hydrograph for the 100-year based on Riverside County Hydrology Manual's recommendations for use with the selected frequency storms (Pg. C-4).

3. **FLOWMASTER -** by Bentley (Haested Methods) was used to estimate underground chamber outflow rates via 12" overflow pipes.

4. **FLOOD HYDROGRAPH ROUTING** – Flood hydrographs routing was used to calculate depth vs storage and depth vs outflow for the underground chambers. Civil Design Software by Bonadiman and Associates was used to model these relationships.
5. **WSPGW** – Water Surface Pressure Gradient Package (WSPGW program) software by Bonadiman and Associates was used for hydraulic calculations.

NARRATIVE

Hydrology-Existing Conditions

The existing 9.1+/- acre site ranges in elevation from approximately 1569 feet at the southwesterly corner to approximately 1532 feet at the northeasterly corner. The site generally sheet flows northeasterly across the site and discharges onto the neighboring property to the east. The discharge continues to sheet flow and is conveyed via street flow toward the Perris Valley Channel.

Watershed “A”

Watershed “A” is defined by the 9.75 acres tributary to the northeast corner of the site.

Watershed “A” generates a 2-year peak flow of 3.6 CFS, a 5-year peak of 5.2 CFS, a 10-year peak flow of 8.9 CFS and a 100-year peak flow of 14.0 CFS as calculated by the Rational Method.

Hydrology-Proposed Conditions

The Thrifty Perris project entails grading the site to accommodate an industrial development. The grading will emulate the existing southwest to northeast drainage pattern while providing an underground infiltration/detention chamber facility along the eastern parking area of the site to treat on-site flows.

On-site flows will be conveyed to this underground facility via proposed on-site storm drains. Runoff produced by the studied storm events will be discharged from 12” overflow pipes placed at various depths of the facility. The flood hydrograph routing analyses for the required storm events demonstrate that flows that will be discharged from the underground facility will be less than pre-developed flows.

In order to perpetuate existing sheet flow drainage conditions for the site and to avoid concentrating flows leaving the site, flows from these overflow pipes will be discharged into a U-shaped channel with flow-over side weirs to discharge these flows as sheet flow as the water moves northerly down the channel. This flow-over side weir system/channel will be located on-site and will not encroach on the adjacent property to the east.

<u>Watershed “A”</u>

Watershed “A” is defined by the 9.0-acre area tributary to the underground chamber facility. Runoff will be discharged into to this facility via two inlet manifolds on each side of the chamber system. Pipes leading from area drains and catch basins around the site will convey site runoff to these inlet manifolds.

Underground Chamber Flood Hydrograph Routing

Flood Hydrograph Routing was used to determine peak flow rates for the overflow pipes serving the underground chamber facility. There are 4 proposed overflow pipes in total. One 12” pipe will be placed at the water level where the Design Capture Volume (DCV) will occur (0.5 depth). The remaining three 12” pipes will be placed at 1.5 ft. above the bottom of the chambers. These peak discharge values leaving the facility are less than those calculated for the existing site using the Rational Method for each respective storm event. The facility drains in less than 72 hours for each storm event.

	PROPOSED FACILITY PEAK OUTFLOW (CFS)			
	2 YR STORM	5 YR STORM	10 YR STORM	100 YR STORM
1 HR DURATION	0.001	0.244	0.711	2.167
3 HR DURATION	0.429	1.319	1.956	4.923
6 HR DURATION	1.299	2.126	2.214	7.704
24 HR DURATION	1.328	1.952	2.163	4.808

In order to perpetuate existing sheet flow conditions, overflow will be released to the County property to the east via a concrete U-shaped channel featuring side weirs along the channel and a grouted rip-rap stabilization blanket.

Peak flow rates leaving each overflow pipe in the controlling 100-year event (6-hour duration) were derived from the flood hydrograph routing runs. The single overflow pipe with the invert placed 0.5 ft. (37.50) from the facility bottom produced a peak flow of 2.9 CFS. Each of the three overflow pipes placed 1.5 ft above the basin bottom (38.50) *each* produced peak flows of 1.6 CFS. From here, the design of the side overflow weirs was calculated for this 100-year controlling event.

Due to the existing property sheet flowing easterly and, the existing flow rate per linear foot under existing conditions was calculated by dividing the Q100 flow rate for the existing, undeveloped site by the length of the eastern property line. With an eastern property line distance of 625' and an existing Q100 of 14.03 CFS, ***the pre-existing undeveloped flow rate per linear foot is 0.022 CFS/FT***. ***The proposed design produces a flow rate per linear foot of 0.019 CFS/FT.*** Therefore, the design of the side weirs guarantees that the proposed flow rate per linear foot ***is less*** than the pre-existing flow rate per linear foot.

The discharge per linear foot of weir was calculated for the u-channels serving each of the three overflow pipes (outlets 1, 2 & 3) at the 38.50 invert elevation. With the initial channel flow of 1.6 CFS, normal depth is 3.3-inches in the channel structure. An initial weir design provides the normal depth of flow would be 0.3 inches above the weir crest elevation along each 10-feet weir segment, the flow per linear foot of weir was calculated to be 0.019 CFS/FT. Thus, the first weir crest elevation is set at 3.0-inches from the channel bottom and is decreased 0.3 inches at the succeeding weir downstream. Approximately 80 feet of weir length is needed to fully distribute this 1.6 CFS of initial discharge along the entirety of the existing ground. ***This is less than the 0.022 CFS/FT under existing conditions.*** With a 10-foot weir segment length, the discharge that would leave the channel via each 10' weir is therefore 0.2 CFS. With each reduction of 0.2 CFS to the overall channel flow per weir, normal depth decreases by 0.3-inches in the channel. Therefore, the downstream weir crest elevation is lowered by 0.3-inches every 10 ft to maintain a constant discharge per linear foot along the entirety of the channel reach.

Downstream of outlets 1 through 3, outlet no. 4 discharges to the channel structure and side weir calculations were also performed for the final reach with its peak flow of 2.9 CFS. For the initial 2.9 CFS design flow in the channel, the normal depth is 2.8 inches. Maintaining the normal depth of flow at 0.3 inches above the weir crest elevation along each 10-foot weir segment, the flow per linear foot of weir was calculated to be 0.019 CFS/FT. Approximately 152 feet of weir length is needed to fully distribute this 1.6 CFS of initial discharge along the entirety of the existing ground as it proceeds down the channel. With a 10-feet weir segment length, the discharge that would leave the channel via each weir is therefore 0.2 CFS. With a reduction of 0.2 CFS to the overall channel flow, normal depth decreases by 0.1 inches in the channel. Therefore, the downstream weir crest elevation is lowered by 0.1 inches to maintain a constant discharge per linear foot along the entirety of the final reach.

Perris Valley MDP Line H10.1 Hydraulic Analysis

In order to determine whether or not Line H-10.1 had sufficient capacity to accept 100-year flows from the project, a hydraulic analysis of Line H-10.1 was performed.

The original design hydraulic conditions of Line H10.1 were modeled using WSPGW and confirmed the hydraulic data as shown in the original Perris Valley MDP Laterals H-10.1 and H-11 drainage report. Due to concerns regarding adequate freeboard for downstream manholes and catch basins, the hydraulic grade line (HGL) of original design conditions and under potential conditions was modeled.

The Rational Method was used to calculate the 100-year peak flow for the proposed, developed site. The Q_{100} for the developed site was calculated to be 21.25 CFS. A point of connection into Lateral H-10.1 was then modeled to create a proposed condition for Lat. H-10.1. Freeboards for downstream and upstream manholes were compared and contrasted for existing and potential conditions. Freeboard values under this 21.25 CFS conditions were not adequate due to calculated water surface elevations being higher than rim elevations at manhole locations. To see if Lateral H-10.1 had any capacity whatsoever, an addition of 5 CFS was modeled instead of 21.25 CFS. This scenario produced inadequate freeboard values as well. Therefore, Lateral H-10.1 does not have any capacity for additional flows from the development. Flows will be routed through the on-site underground detention facility and will exit through overflow pipes. These flows will be dispersed via sheet flow through a proposed U-shaped channel with side weirs to perpetuate existing sheet flow conditions of the watershed. Please see Appendix H for hydraulic calculations.

Half-Street and Offsite Flows

The Rational Method was used to calculate peak flows for the half-street of Water Street and Tobacco Road adjacent to the proposed project. The 10-year and 100-year events were considered in this analysis. The 10-year and 100-year peak flow for Water Street was calculated to be 0.82 CFS and 0.95 respectively. The 10-year and 100-year peak flow for Tobacco Road was calculated to be 0.72 CFS and 1.02 respectively.

Offsite flows and planned storm drain improvements near the project site were obtained from the RCFC/WCD Perris Valley Area Drainage Plan. Per this plan, a proposed 36” CMP Riser Inlet connected to a proposed 36” storm drain on Water Street (Lateral H-10) will convey offsite flows from the southwest to an existing 36” RCP near the southeast corner of the site. This drainage plan has been provided in Appendix G.

Conclusion

The narrative and supporting calculations in this report support the conclusion that the proposed site design and on-site detention facility mitigate the impacts of increased flows caused by development. Flows will continue to be discharged in a manner that reflects and perpetuates the sheet flow behavior of the existing site. In addition, the inability to add additional flows from the developed site into existing Lateral H-10.1 on Placentia Avenue has also been supported by relevant calculations and analysis.

Appendix "A" - Existing Conditions Calculations