Los Molinos Drainage Study

Hydraulic Assessment of Primary Drainage Facilities

Tehama County

Prepared for:

Tehama County Department of Public Works
9380 San Benito Avenue
Gerber, CA 96035

Prepared by:

Pacific Hydrologic Incorporated
1050 West Street
Redding, CA 96001

Norman S. Braithwaite, P.E.

December 31, 2007

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INTRODUCTION

The Town of Los Molinos experiences shallow flooding on a relatively frequent basis. The sources and rough extents of flooding have been studied and documented the report "Los Molinos Drainage Study, Existing Condition Flood Hydrology, Tehama County" dated December 13, 2007. Subsequent to preparation of this report, Tehama County has prepared a preliminary drainage facility plan and has requested hydraulic assessment of the primary facilities. This study and report includes flood hydrology for, and hydraulic assessment of the primary drainage facilities during a 50-year flood in the Sacramento River.

PRELIMINARY DRAINAGE PLAN

The preliminary drainage plan identified by Tehama County includes the following features:

- Prevent overflow from Los Molinos Creek from entering the developed area of Los Molinos.
- Construct new storm drainage facilities to collect runoff north of Grant Street and convey it to Mill Slough using a new storm drain pipe under State Route 99E (SR-99E) and the Union Pacific Railroad (UPRR) tracks.
- Construct new storm drainage facilities to collect runoff between Grant Street and South Center Street and convey it to a new open channel leading to Mill Slough using a new storm drain pipe under SR-99E and the UPRR tracks.
- Construct new storm drainage facilities to collect runoff between South Center Street and Lee Street and convey it to a new open channel leading to Mill Slough using a new storm drain pipe under SR-99E and the UPRR tracks.
- Modify the Los Molinos Mutual Water Company (LMMWC) crossing under the UPRR tracks to better accommodate winter storm drainage.

Overflow from Los Molinos Creek will be prevented using a combination of channel improvements, earth berms, modification of the LMMWC irrigation ditch crossing of Los Molinos Creek, and, if necessary, bridge improvements. The three proposed storm drainage facilities described above are shown on Figure 1 as "North", "Center", and "South" facilities along with their contributing drainage basins. A small area located south of Los Molinos Creek presently draining to the LMMWC irrigation ditch is assumed to continue draining to the irrigation ditch. Overflow from Champlin Slough which is presently believed to flow in a southerly direction parallel to the slough are assumed to be confined to the Champlin Slough floodplain and not enter the more densely developed areas in the Town of Los Molinos. The area located south of Grant Street between the UPRR tracks and SR-99E presently drains to a low lying area with no apparent outlet. At present the preliminary drainage plan does not include facilities to drain this low lying area.

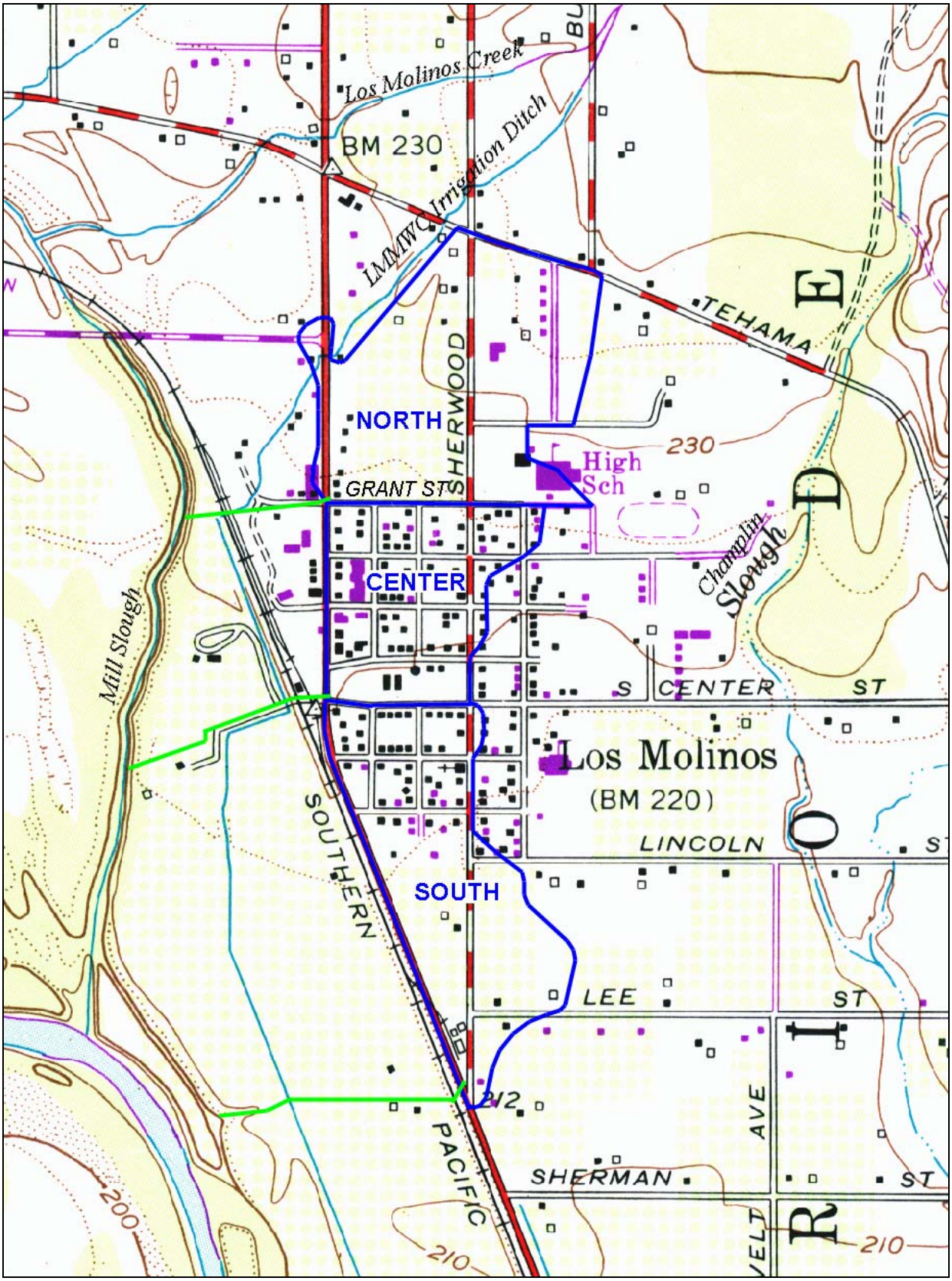

Figure 1: Los Molinos Drainage Basins and Primary Drainage Facilities

The primary drainage facilities addressed by this study are identified on Figure 1 by the green lines between the identified drainage basins and Mill Slough. The preliminary drainage plan identifies these facilities as follows:

- North – 1280-feet of 48-inch CMP and metal pipe with a downstream invert elevation of 216.0-feet and an upstream invert elevation of 218.0-feet.
- Center – 680-feet of 48-inch CMP and 600-feet of open channel. The upstream invert elevation is identified as 214.8-feet. The channel is identified as having a bottom width of 3.0-feet and side slopes of 2H:1V in a 25-foot wide easement.
- South – Approximately 110-feet of 60-inch CMP and approximately 1890-feet of open channel.

Water surface elevations in the Sacramento River are estimated to be 214-, 213-, and 210-feet at the outfalls of these three new facilities respectively. Although the outfalls will be to Mill Slough, large floods in the Sacramento River inundate Mill Slough.

HYDROLOGIC ANALYSIS

The peak flow during a 100-year storm was estimated for each of the three new facilities by modifying a copy of the rainfall-runoff model prepared for the existing condition flood hydrologic analysis and rerunning. A schematic of the rainfall-runoff model modified to reflect the preliminary drainage study facilities is shown in Figure 2.

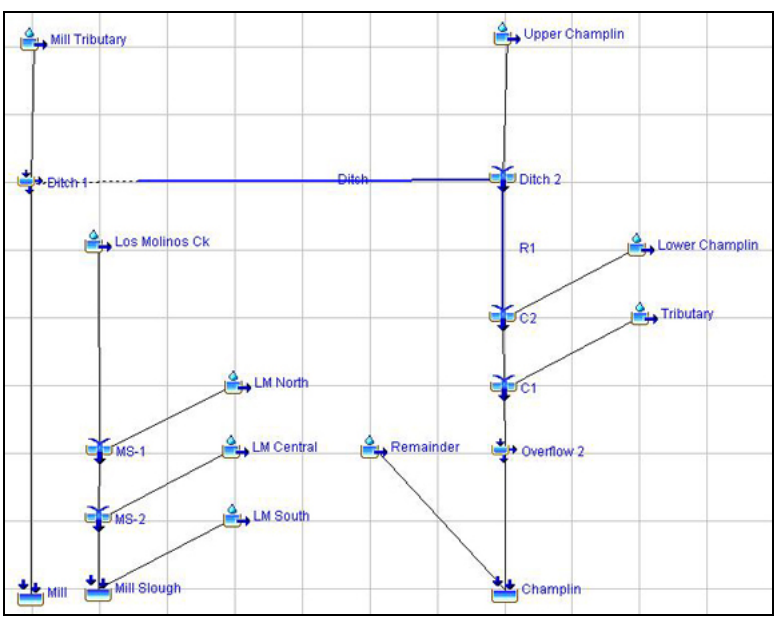

Figure 2: Revised Rainfall-Runoff Model Subbasin Schematic

Abstraction data for the new basins were estimated from abstraction data identified for the "developed area" subbasin of the existing condition rainfall-runoff model and from a recent aerial photograph. Area weighted curve numbers and initial abstractions were estimated as described in the existing condition flood hydrology report. The curve numbers and abstraction data initially determined and used in the model are summarized in Table 1.

Table 1
Area Weighted Curve Numbers and Initial Abstractions

Subbasin	AMC-II (from abstraction data)		AMC-III (used in model)	
	Curve Number	Initial Abstraction	Curve Number	Initial Abstraction
North	71	0.817	86	0.326
Center	71	0.817	86	0.326
South	69	0.899	84	0.381

Subbasin lag data were roughly estimated by experience to be 20-, 15-, and 30-minutes for the north, center, and south basins respectively. Actual subbasin lag times in these basins will be a function of the efficiency of the new drainage facilities.

Design storm precipitation data developed for the existing condition flood hydrologic analysis were also used in the rainfall-runoff model prepared to identify the requirements of the primary drainage facilities.

Results of the rainfall-runoff model are summarized in Table 2.

Table 2
Peak Flows From 100-Year Storm

Subbasin	100-Year Peak Flow (cfs)
North	151
Center	115
South	84

HYDRAULIC ANALYSIS AND RESULTS

Hydraulic evaluation of primary drainage facilities was accomplished by conducting trial and error hydraulic calculations using a spreadsheet. The headwater elevation necessary to convey the peak flow estimated during the 100-year storm was calculated for culverts assuming submerged outlet control conditions during a 50-year flood in the Sacramento River and considering appropriate gradient for the open channel sections between the culverts and Mill Slough. The required headwater was then compared to the ground elevations at the upstream end of the culvert to determine if the culvert was capable of conveying the arriving flow or not. If the initially identified primary drainage facility was not capable of conveying the peak flow estimated during the 100-year storm, other pipe types and sizes were tested until an acceptable pipe was found. Entrance and outlet coefficients were assumed to be 0.4 (square edge) and 1.0 velocity heads respectively. Manning's roughness coefficients of 0.024 and 0.013 were used to represent CMP and smooth (steel or concrete) pipe respectively. The slope of open channels was initially assumed to be 0.2-percent (decimal 0.002). At the south facility, however, this slope produced a water surface elevation at the upstream end of the open channel in excess of the ground elevation at State Route 99E. Therefore, the slope of the open channel for this facility was reduced to 0.1-percent (decimal 0.001). The culvert calculations are summarized in Table 3. A printed copy of the spreadsheet is included in Appendix A.

Table 3
Summary of Culvert Calculations

Location & 100-year flow	Diameter (inches)	Material	Tailwater Elev (feet)	Req'd Head -water (ft)	Ground Elev (feet)	Condition
North, 151-cfs	48	CMP	214	265.2	224.5	Inadequate
	48	Smooth	214	231.2	224.5	Inadequate
	54	Smooth	214	223.2	224.5	OK
Center, 115-cfs	48	CMP	214.2	230.8	220.0	Inadequate
	48	Smooth	214.2	220.4	220.0	Inadequate
	54	Smooth	214.2	217.5	220.0	OK
South, 84-cfs	60		213.8[1]		212.5	Inadequate
	60	CMP	211.9[2]	213.0	212.5	Inadequate
	60	Smooth	211.9[2]	212.5	212.5	OK

Notes: 1) 0.2-percent energy slope assumed for downstream open channel. Water surface elevation at upstream end of open channel exceeds elevation of State Route 99E.
2) 0.1-percent energy slope assumed for downstream open channel.

After identifying adequate culverts, the dimensions of open channels were estimated by trial and error using a spreadsheet. Manning's roughness for the open channels was assumed to be 0.040, representative of small size RSP and managed vegetation. Side slopes of 2H:1V were assumed for all open channels. Energy slopes of 0.2- and 0.1-percent were assumed for channels downstream of the center and south drainage areas respectively. The culvert calculations are summarized in Table 4. A printed copy of the spreadsheet is included in Appendix A.

Table 4
Summary of Open Channel Calculations

Location & 100-year flow	Base (feet)	Slope	Depth (feet)	Capacity (cfs)	Condition
Center, 115-cfs	3	.002	3.1	123	OK
South, 84-cfs	3	.001	3.1	87	OK

OTHER CONSIDERATIONS

Upstream Drainage: Unless designed considering surcharged conditions, during surcharged conditions of the primary drainage facilities, the upstream contributing drainage facilities will also surcharge. When this happens, flood water is stored in connected low lying areas and, if water surface elevations get high enough, conveyed in other directions as shallow overflow. Therefore, some ponding and shallow overflow may occur during large local storm events when water surface elevations in the Sacramento River are high.

Culvert Outlets: Energy dissipaters will be necessary to prevent erosion at the outlet of the culverts. The velocity of water in the culverts is likely to be higher during storm/flood combinations of lesser magnitude than the preliminary design condition.

Open Channel Invert Elevations: The invert elevations of open channels must be set at least 3.1-feet below the hydraulic grade line for open channels. The hydraulic grade line for the open channel draining the center facility is 214.2-feet at the culvert outlet and decreasing at a slope of 0.2-percent to an elevation of approximately 213.0-feet at Mill Slough. The hydraulic grade line for the open channel draining the south facility is 211.9-feet at the culvert outlet and decreasing at a slope of 0.1-percent to an elevation of approximately 210.0-feet at Mill Slough.

Open Channel Top Width: Although the width of the channel at the preliminary design water surface elevation will be approximately 15-feet, the total width of the channel will be determined by the depth of the channel with respect to the surrounding ground and the geometry of the channel above the design water surface elevation.

Culvert-Open Channel Conform: This preliminary design has been based on energy grade line calculations with submerged outlet control assumed for the culverts. As such, the minimum (highest) calculated invert of the open channels will be higher than the invert of the culverts feeding the channels and conform sections will be necessary at the upstream end of the channels. Construction of the primary drainage facilities in this manner may result in deposition of sediment in conforms and a requirement of periodic cleaning. Alternatively, the invert of the open channels may be lowered resulting in higher channel construction costs if the energy grade line is maintained as identified, or greater deposition may occur in the channels if the energy gradient is reduced. Use of multiple smaller diameter culverts having an equal total capacity may also avoid the disparity in invert elevations at conforms.

Maintenance: The velocity of water flowing in the smooth walled culverts during larger floods should be sufficient to prevent accumulation of sediments in the bottom of the culverts. Maintenance of the hydraulic capacity of the open channels will require management of vegetation. Acceptable vegetation includes grasses and reeds that will lean over in water moving at three-feet per second. Unacceptable vegetation includes blackberry, Arundo, and trees or large shrubs having trunks in the flood channel. The velocity of water flowing in the open channels will not be sufficient to prevent deposition of sediment. Therefore, unless the open channels are oversized or constructed with a paved bottom, occasional removal of deposited sediment will be necessary to maintain hydraulic capacity. Given the low gradient topography of the contributing basins, the nature of sediment is expected to be silt, sand and limited volumes of gravel (primarily from imported sources).

RECOMMENDATIONS

Design of Primary Drainage Facilities: Complete hydraulic design calculations, including entrance control conditions, should be conducted as part of the process to select and identify primary drainage facilities to be constructed. Upstream contributing facilities should be designed considering surcharged conditions at the primary drainage facilities or overflow should be expected and accommodated in road sections during the design condition.

A detailed backwater analysis of Los Molinos Creek is recommended to identify improvements necessary to prevent overflow during the most probable 100-year storm.

APPENDIX A

Spreadsheet Calculations

Los Molinos Drainage Study
Drainage System Requirements during Q50 on Sacramento River
December 26, 2007

Primary Culverts:

					Submerged Outlet Control								
Description	Q (cfs)	D (feet)	L (feet)	V (fps)	Ke	n	H	TW	HW	DI Invert	Ground	Condition	Notes
Grant St	151	4.00	1280	12.0	0.4	0.024	51.176	214	265.2	218.0	224.5	*Surcharge*	CMP, TW = FEMA E50
	151	4.00	1280	12.0	0.4	0.013	17.238	214	231.2	218.0	224.5	*Surcharge*	Concrete or Smooth Steel, TW = FEMA E50
	151	4.50	1280	9.5	0.4	0.013	9.2009	214	223.2	218.0	224.5	OK	Concrete or Smooth Steel, TW = FEMA E50
S Center St	115	4.00	680	9.2	0.4	0.024	16.624	214.2	230.8	214.8	220.0	*Surcharge*	CMP, TW = FEMA E50 + 1.2-feet (0.002 ditch slope)
	115	4.00	680	9.2	0.4	0.013	6.1666	214.2	220.4	214.8	220.0	*Surcharge*	Concrete or Smooth Steel, TW = FEMA E50 + 1.2-feet (0.002 ditch slope)
	115	4.50	680	7.2	0.4	0.013	3.2915	214.2	217.5	214.8	220.0	OK	Concrete or Smooth Steel, TW = FEMA E50 + 1.2-feet (0.002 ditch slope)
Lee Street	84	5.00	240	4.3	0.4	0.024	1.1447	213.8	214.9		212.5	*Surcharge*	CMP, TW = FEMA E50 + 3.8-feet (0.002 ditch slope)
	84	5.00	240	4.3	0.4	0.024	1.1447	211.9	213.0		212.5	*Surcharge*	CMP, TW = FEMA E50 + 1.9-feet (0.001 ditch slope)
	84	5.00	240	4.3	0.5	0.013	0.5664	211.9	212.5		212.5	OK	Concrete or Smooth Steel, TW = FEMA E50 + 1.9-feet (0.001 ditch slope)

Downstream Ditches:

Description	B (feet)	Slope	H:V	n	Assumed D (feet)	A	Rh	V	Cap (cfs)	Top (ft)	Q (cfs)	Condition	Notes
S Center St	3	0.002	2	0.04	3.1	28.5	4.1	4.3	123	15.4	115	OK	Narrow ditch
	4	0.002	2	0.04	2.8	26.9	4.3	4.4	119	15.2	115	OK	Wide ditch
Lee Street	3	0.001	2	0.04	3.1	28.5	4.1	3.0	87	15.4	84	OK	Narrow ditch
	4	0.001	2	0.04	2.8	26.9	4.3	3.1	84	15.2	84	OK	Wide ditch

APPENDIX B

References

REFERENCES

1) Design and Construction of Urban Stormwater Management Systems, Urban Water Resources Research Council of the American Society of Civil Engineers, ASCE Manuals and Reports of Engineering Practice 77, ASCE, 1992

2) Los Molinos Drainage Study, Existing Condition Flood Hydrology, Tehama County (unpublished report), Pacific Hydrologic Incorporated, December 13, 2007