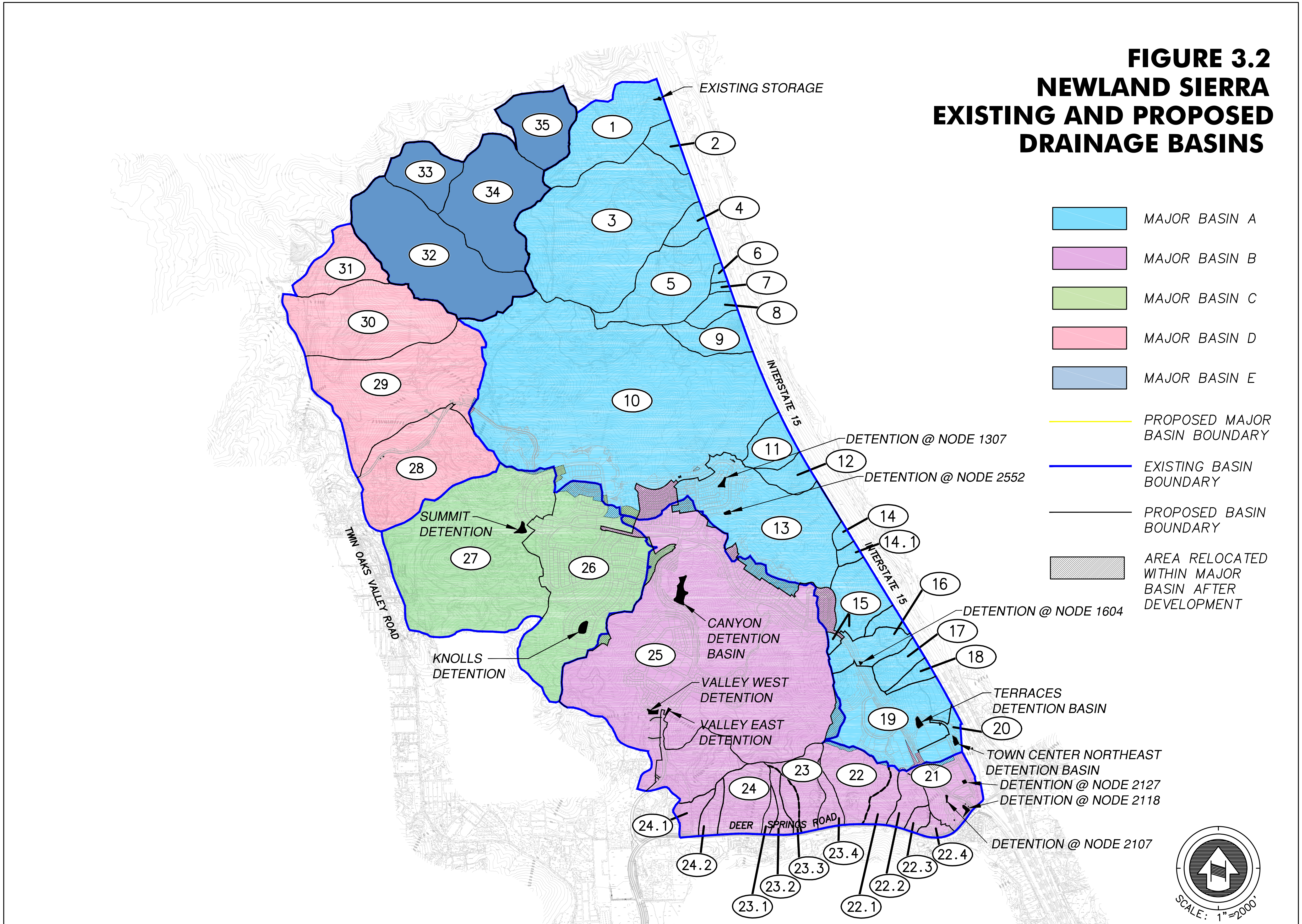
FIGURE 3.2
NEWLAND SIERRA
EXISTING AND PROPOSED
DRAINAGE BASINS



M:\PROJECTS\2660 NEWLAND COMMUNITIES\002-02 NEWLAND SIERRA\ENG\STORMWATER\DRAINAGE STUDY\MAIN REPORT\FIG 3.2 - EXISTING & PROPOSED DRAINAGE BASIN EXHIBIT.DWG (02-24-17 2:25:06PM) Plotted by: rroberts

3.2 IMPACTS OF PROPOSED PROJECT ON WATERSHEDS

3.2.1 SOUTH FORK MOOSA CANYON WATERSHED

South Fork Moosa Canyon watershed was divided into 22 subbasins to analyze the runoff to the culverts and storm drain systems that convey water from the west side to the east side of Highway I-15. These original basins were created based on knowledge available at the time of culvert locations. Later research revealed more than 22 inlets leading to storm drain systems under Highway I-15. In some cases, multiple inlets confluence to a single outlet, furthermore, drainage from Highway I-15 can also be included in these systems.

Impacts downstream were first evaluated by comparing storm drain flows for Existing and Proposed conditions (see Table 3.1.4a). Where proposed conditions led to an increase in runoff due to development, onsite detention basins were utilized in order to mitigate peak flows. Taking into account the mitigation measures, there were no remaining appreciable increases in storm drain outfall flows. Additional details for each basin are provided in the discussion below.

Table 3.2.1 South Fork Moosa Canyon Drainage Basin

<table>
<tr><td colspan="5">As-Built Conditions</td><td></td><td colspan="2">Pre-development</td><td colspan="2">Post-development</td></tr>
<tr><td>Caltrans As-Built Contract No.</td><td>Caltrans System</td><td>Caltrans Station</td><td>Designed Capacity (cfs)</td><td colspan="2">Current Pipe Dia (in)</td><td>Project Sub-basin</td><td>Undetained Existing Peak Flow Rate (cfs)</td><td>Area (AC)</td><td>New Post-Detention Peak Flow Rate (cfs)</td><td>Area (AC)</td></tr>
<tr><td>11-095054</td><td>24</td><td>1917 ±</td><td>174</td><td colspan="2">48</td><td>1</td><td>87</td><td>71.9</td><td>87</td><td>71.9</td></tr>
<tr><td>11-095054</td><td>22</td><td>1905 ±</td><td>27</td><td colspan="2">30</td><td>2</td><td>27.10</td><td>13.2</td><td>27.10</td><td>13.2</td></tr>
<tr><td>11-095054</td><td>20</td><td>1897 ±</td><td>601</td><td colspan="2">84</td><td>3</td><td>217.44</td><td>199.7</td><td>217.44</td><td>199.7</td></tr>
<tr><td>11-095054</td><td>18</td><td>1893 ±</td><td>35</td><td colspan="2">30</td><td>4</td><td>14.00</td><td>11.6</td><td>14.00</td><td>11.6</td></tr>
<tr><td>11-095054</td><td>15</td><td>1884 ±</td><td>71</td><td colspan="2">30/36</td><td>5.1</td><td>20.77</td><td>19.9</td><td>20.77</td><td>19.9</td></tr>
<tr><td>11-095055</td><td>14</td><td>1883 ±</td><td>140</td><td colspan="2">36/42</td><td>5</td><td>52.64</td><td>49.0</td><td>52.64</td><td>49.0</td></tr>
<tr><td>11-095054</td><td>12</td><td>1875+50</td><td>6</td><td colspan="2">24</td><td>6</td><td>5.27</td><td>2.5</td><td>5.27</td><td>2.5</td></tr>
<tr><td>11-095054</td><td>11</td><td>1873 ±</td><td>4</td><td colspan="2">24</td><td>7</td><td>3.89</td><td>2.1</td><td>3.89</td><td>2.1</td></tr>
<tr><td>11-095054</td><td>10</td><td>1971+50</td><td>48</td><td colspan="2">30</td><td>8</td><td>16.15</td><td>11.7</td><td>16.15</td><td>11.7</td></tr>
<tr><td>11-095054</td><td>9</td><td>1864+50</td><td>70</td><td colspan="2">42</td><td>9</td><td>30.46</td><td>25.8</td><td>30.46</td><td>25.8</td></tr>
<tr><td>11-095054</td><td>7</td><td>1855 ±</td><td>1218</td><td colspan="2">42</td><td>10</td><td>783</td><td>486.9</td><td>762</td><td>474.3</td></tr>
<tr><td>11-095054</td><td>4</td><td>1840 ±</td><td>70</td><td colspan="2">42</td><td>11</td><td>43</td><td>21.7</td><td>43</td><td>21.6</td></tr>
<tr><td>11-095054</td><td>2</td><td>1833 ±</td><td>--</td><td colspan="2">36</td><td>12</td><td>23.36</td><td>14.2</td><td>23.36</td><td>14.2</td></tr>
<tr><td>11-095044</td><td>27</td><td>1820 ±</td><td>396</td><td colspan="2">36</td><td>13</td><td>199</td><td>130.0</td><td>177</td><td>142.6</td></tr>
<tr><td>11-095044</td><td>26</td><td>1815 ±</td><td>11</td><td colspan="2">24</td><td>14</td><td>6.36</td><td>3.9</td><td>6.36</td><td>3.9</td></tr>
<tr><td>11-095044</td><td>25</td><td>1810 ±</td><td>8</td><td colspan="2">24</td><td>14.1</td><td>6.87</td><td>4.4</td><td>6.87</td><td>4.4</td></tr>
<tr><td>11-095044</td><td>24</td><td>1806 ±</td><td>20</td><td>24</td><td rowspan="2">48</td><td rowspan="2">15</td><td rowspan="2">50.75</td><td rowspan="2">43.3</td><td rowspan="2">47.12</td><td rowspan="2">36.20</td></tr>
<tr><td>11-095044</td><td>23</td><td>1800 ±</td><td>84</td><td>36</td></tr>
<tr><td>11-095044</td><td>21</td><td>1790 ±</td><td>112</td><td colspan="2">42</td><td>16</td><td>41</td><td>29.6</td><td>41</td><td>23.8</td></tr>
<tr><td>11-095044</td><td>20</td><td>1785 ±</td><td>42</td><td colspan="2">30/36</td><td>17</td><td>20.87</td><td>14.3</td><td>20.87</td><td>14.3</td></tr>
<tr><td>11-095044</td><td>19</td><td>1780 ±</td><td>20</td><td colspan="2">24/30</td><td>18</td><td>8.36</td><td>6.5</td><td>8.36</td><td>6.5</td></tr>
<tr><td>11-095044</td><td>18</td><td>1775 ±</td><td>243</td><td colspan="2">60</td><td>19</td><td>93</td><td>81.2</td><td>81</td><td>95.0</td></tr>
<tr><td>11-095044</td><td>16</td><td>1768 ±</td><td>34</td><td colspan="2">30</td><td>20</td><td>26</td><td>41.3</td><td>12</td><td>9.9</td></tr>
</table>

Subbasins with Significant Increase in Runoff (Prior to Mitigation)

<u>Subbasin 13</u> Development is planned for the upper portion of Subbasin 13, leading to an increase in runoff production from this area. In order to maintain 100-year peak flows from this subbasin at or below the existing conditions, a pair of detention basins is proposed in Subbasin 13. Under existing conditions, the modeled peak flow in Subbasin 13 was 199 cfs. Under developed conditions, assuming no mitigation, the peak flow from Subbasin 13 at the discharge point is 214 cfs. In order to mitigate the increase in peak flow, detention basins are proposed at Nodes 1307 and 2552. With these detention basin included in the model, the peak flow from Subbasin 13 is 177 cfs.

Caltrans calculations show a design flow capacity of 396 cfs in the 36 inch pipe, far in excess of the anticipated flow under proposed conditions.

<u>Subbasin 16</u> Road improvement work proposed within Subbasin 16 leads to a slight increase in anticipated runoff production from this area. In order to maintain 100-year peak flows from this subbasin at or below the existing conditions, a detention basin is proposed in Subbasin 16 along the proposed roadway. Under existing conditions, the modeled peak flow in Subbasin 16 was 41 cfs. Under developed conditions, assuming no mitigation, the peak flow from Subbasin 16 at the discharge point is 47 cfs. In order to mitigate the increase in peak flow, a detention basin is proposed adjacent to the proposed roadway at Node 1604. With this detention basin included in the model, the peak flow from Subbasin 16 is 41 cfs.

<u>Subbasin 19</u> Development is planned for Subbasin 19, leading to a large increase in runoff production. In order to maintain 100-year peak flows from this subbasins at or below the existing conditions, a detention basin is proposed in Subbasin 19. Under existing conditions, the modeled peak flow in Subbasin was 93 cfs. Under developed conditions, assuming no mitigation, the peak flow from Subbasin 19 into the culvert is 219 cfs. In order to mitigate the increase in peak flow, a detention basin is proposed adjacent to the proposed school site. With this detention basin included in the model, the peak inflow into the culvert (and therefore from Subbasin 19) is 74 cfs.

<u>Subbasin 20</u> Development is planned for Subbasin 20, leading to a large increase in runoff production. In order to maintain 100-year peak flows from this subbasins at or below the existing conditions, a detention basin is proposed in Subbasin 20. Under existing conditions, the modeled peak flow in Subbasin 20 was 26 cfs. Under developed conditions, assuming no mitigation, the peak flow from Subbasin 20 at the discharge point is 43 cfs. In order to mitigate the increase in peak flow, a detention basin is proposed adjacent to the proposed development. With this detention basin included in the model, the peak inflow into the culvert (and therefore from Subbasin 20) is 18 cfs.

Subbasins with No Increase in Runoff

Subbasin 1 Subbasin 2 Subbasin 3 Subbasin 4 Subbasin 5 Subbasin 6 Subbasin 7 Subbasin 8 Subbasin 9 Subbasin 10 Subbasin 11 Subbasin 12 Subbasin 14 Subbasin 14.1 Subbasin 15 Subbasin 15.1 Subbasin 17 Subbasin 18	No development is planned for Subbasins 1-9,11, 12, 14, 14.1, 15.1, 17 and 18. Anticipated peak flow values therefore remain the same (or are reduced due to reductions in drainage area) between existing and proposed conditions. In Subbasin 10, the proposed basin area is slightly less than the existing basin area, and only a small portion of the basin is developed. Based on hydrology routing, there is no increase in runoff from Subbasin 10 due to development. Existing culvert sizes are adequate to safely convey flow from these subbasins.

3.2.2 SAN MARCOS CREEK WATERSHED

San Marcos Creek watershed is composed of two basins, Basin B and Basin C. These basins were broken up into a total of 17 subbasins in order to determine the rate of flow to each culvert under Deer Springs Road. In order to evaluate the downstream impacts of any post-developed runoff, two conditions were considered: any increase of maximum runoff rates (Q) between existing and proposed conditions, and the Q to each culvert under Deer Springs Road.

Subbasins with Significant Increase in Runoff (Prior to Mitigation)

Subbasin 21	Significant commercial development is planned for Subbasin 21, which leads to an increase in runoff. Under existing conditions, the modeled peak flow discharging from Subbasin 21 was 42 cfs. Under developed conditions, without any mitigation, the modeled peak flow was 131 cfs. In order to mitigate the peak flow, three detention basins are proposed within Subbasin 21. With the detention basins included, under proposed conditions the modeled peak flow was 41 cfs. Detention basins are proposed at hydrology nodes 2107.5 (BMP TCR2), 2118.6 (BMP TCR3), and 2127 (BMP

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TC4). The basins at nodes 2107.5 (BMP TCR2) and 2118.6 (BMP TCR3) are surface detention basins which primarily accept runoff from the public roadways within the subbasin. Basin 2107.5 (BMP TCR2) will have an 8″ orifice outlet, while basin 2118.6 (BMP TCR3) will have an 18″ orifice outlet. Both basins will also function for pollutant control and hydromodification, so the orifice outlets will be raised 1.0′ above the bottom of the basin. The detention facility at node 2127 (BMP TC4) is located in the commercial area of the Town Center. This detention facility is conceptually modeled as an underground vault, with a footprint of 10,000 sf and a depth of five feet. Outflow will be controlled by a 6″ orifice. See Section 3.1.2 for discussion of proposed detention basin analysis. Details of storage routing analysis and a plan view of the detention basin are included in Appendix B.

<u>Subbasin 25</u> Basin 25 consists of 541 acres under existing conditions, with a peak flow of 677 cfs. Under proposed conditions, the peak flow increases to 836 cfs. A pair of detention basins, Valley West (BMP V11VR2) and Valley East (BMP V14), will flank the entrance road to the Valley Neighborhood, and depression storage will be utilized as a detention basin in the norther canyon (BMP M1MR1K1) with Basin 25. These basins will reduce the flows offsite to 634 cfs. This not only maintains the existing conditions, it reduces the peak flow to reduce downstream impacts. See Section 3.1.2 for discussion of proposed detention basin analysis. Details of storage routing analysis and a plan view of the detention basin are included in Appendix B.

<u>Subbasin 26</u> Major Basin C has two subbasins, Subbasins 26 and 27. Subbasin 26 has extensive development, and a detention basin will be constructed near the outlet of the developed area to limit flows and downstream impacts. Under existing conditions, the runoff peak is 193 cfs, which increases to 274 cfs under proposed conditions. A detention basin, Detention 26 (BMP K8), has been designed to reduce the peak outflow to 170 cfs. This prevents downstream impacts and will serve as a water quality device for the development as well.

See Section 3.1.2 for discussion of proposed detention basin analysis. Details of storage routing analysis and a plan view of the detention basin are included in Appendix B.

<u>Subbasin 27</u> Subbasin 27 has some development planned, and a detention basin

will be constructed downstream of the developed area to limit flows and downstream impacts. Under existing conditions, the runoff peak is 230 cfs, which increases to 236 cfs under proposed conditions. A detention basin, Detention 27 (BMP S2), has been designed to reduce the peak outflow to 185 cfs. This prevents downstream impacts and will serve as a water quality device for the development.

See Section 3.1.2 for discussion of proposed detention basin analysis. Details of storage routing analysis and a plan view of the detention basin are included in Appendix B.

Subbasin 22 — Subbasin 22 is located along the hillside north of Deer Springs Road, and has an existing area of 38.4 acres. It contains a water quality basin that is used to treat approximately 2,900 linear feet so the

The size of the basin increases to 42.0 acres as it includes a water quality basin to treat 2,900 linear feet of Deer Springs Road's runoff. It also includes the full width of the expanded road, whereas the existing conditions ended at the northern right of way. As expected with such an increase, the peak flows increase as well, from an existing peak of 75 cfs to a proposed peak of 97 cfs. In order to mitigate for the increased runoff, the water quality basin was deepened to provide peak flow detention as well. Post-detention, the peak is reduced to 80 cfs, an increase of 5 cfs from the existing condition. Taken all together, Subbasins 22, 23.4 and 23 see a total increase of 12 cfs, which is offset from the upstream basins (22.4, 22.3, 22.2, 22.1) seeing a total decrease of 12 cfs.

Subbasin 23.4 — Subbasin 23.3 increases in size from the existing due to it having a portion of Deer Springs Road draining to it to be treated in the water quality/peak flow detention basin. It showed an undetained flow increase from 16 cfs to 25 cfs. Post-detention it is decreased to 20 cfs, an increase of 4 cfs. Taken all together, Subbasins 22, 23.4 and 23 see a total increase of 12 cfs, which is offset from the upstream basins (22.4, 22.3, 22.2, 22.1) seeing a total decrease of 12 cfs.

Subbasin 23 — Subbasin 23 increases in size from the existing due to it having a portion of Deer Springs Road draining to it to be treated in the water quality/peak flow detention basin. It showed an undetained peak flow increase from 50 cfs to 67 cfs. Post-detention that peak is reduced to 53 cfs, an increase of 3 cfs. Taken all together, Subbasins 22, 23.4 and 23 see a total increase of 12 cfs, which is offset from the upstream basins (22.4, 22.3, 22.2, 22.1) seeing a total decrease of 12 cfs.

Subbasin 24 — Subbasin 24 increases in size from the existing due to it having a portion of Deer Springs Road draining to it to be treated in the water quality/peak flow detention basin. It showed an undetained peak flow increase from 79 cfs to 88 cfs. Post detention, that peak is reduced to 79 cfs, thus matching the existing conditions.

Subbasins with No Increase in Runoff

Subbasin 22.4, 22.3, 22.2, 22.1 — These subbasins all decreased in size due to the road surface being routed to sub-basin 22 to be treated for water quality. Subbasin 22.4 sees a peak flow decrease of 1 cfs, subbasin 22.3 sees no change, subbasin 22.2 decreases by 9 cfs, and subbasin 22.1 decreases by 2 cfs. All together, the subbasins upstream of subbasin 22 decreased by 12 cfs.

Subbasin 23.3 — Subbasin 23.3 has a slight decrease in size due to it being cut off by a water quality/peak flow detention basin for subbasin 23. The decrease in size was not enough to register a significant decrease in the 18 cfs of peak flow runoff from the existing conditions. It is simply passed under the road as it was in the existing conditions.

Subbasin 23.2 — Subbasin 23.2 sees a slight decrease in area from 7.9 ac to 7.2. This is due to the adjacent road draining to the water quality basin in subbasin 23. As a result it sees a decrease in peak flow from 18 cfs to 17 cfs, a total decrease of 1 cfs.

Subbasin 23.1 — Subbasin 23.1 sees a slight decrease in area from 4.5 ac to 4.2 ac. This is due to the adjacent road draining to the water quality basin in subbasin 23. The decrease in size was not enough to register a significant decrease in the 10 cfs of peak flow runoff from the existing conditions. It is simply passed under the road as it was in the existing condition.

Subbasin 24.2 — Subbasin 24.2 sees a slight decrease in area from 9.0 ac to 8.3 ac. This is due to the adjacent road draining to the water quality basin in subbasin 24. That same basin protrudes into the sub-basin 24.2 which also contributes to the decreased area. As a result of these decreases in area, the subbasin has a decrease in peak flow from 19 cfs to 18 cfs, a difference of 1 cfs.

Subbasin 24.1 In the existing condition subbasin 24.1 flows westerly along the northern edge of Deer Springs Road and/or overtops the road during larger storm events. In the proposed condition, it is picked up and transmitted in the "clean" storm drain line where it confluences with the Stevenson Creek box culvert at the intersection of Deer Springs Road and Sarver Lane. It's area is counted in the "Offsite Improvements" subbasin 34.9.

3.2.3 SOUTH FORK GOPHER CANYON WATERSHED

South Fork Gopher Canyon watershed is divided into two major basins, Basin D and E. Basin D is comprised of Subbasins 28 through 31, and Basin E is comprised of Subbasins 32 through 35.

Subbasins with Significant Increase in Runoff (Prior to Mitigation)

<u>Subbasin 28</u> Subbasin 28 is located along a proposed road, Camino Mayor. In the existing conditions there is 131 cfs of runoff. In the proposed condition that will increase to 133 cfs. To reduce this increase, a detention basin is proposed alongside the roadway, which will reduce the peak flow to 127 cfs.

<u>Subbasin 29</u> Development in Subbasin 29 is limited to the upper reaches of the drainage area, which contains portions of the equestrian staging area and Camino Mayor. In existing conditions, the peak flow from this basin is 226 cfs. The additional impervious area in the basin would increase the flow rate to 229 cfs. To more closely match the existing condition, a detention basin is proposed near the equestrian staging area. This detention basin will reduce the peak flow to 227 cfs. This is a negligible increase (less than 0.5%) over the existing flow rate. Additional, when taken in combination with Subbasin 28, there is a net decrease in the peak flow rate discharged to the South Fork of Gopher Canyon.

Subbasins with No Increase in Runoff

<u>Subbasin 30</u>
<u>Subbasin 31</u>
<u>Subbasin 32</u>
<u>Subbasin 33</u>
<u>Subbasin 34</u>
<u>Subbasin 35</u>

No development is planned for Subbasins 30-34. Anticipated peak flow values therefore remain the same between existing and proposed conditions.

<u>3.3 OFFSITE FLOODPLAIN ANALYSIS</u>

To analyze potential impacts from the project's offsite improvements to FEMA mapped floodplains and floodways, a separate study has been prepared. For more information, please refer to the Floodplain Revision and No-Rise Report for Stevenson Creek and Twin Oaks Valley Creek.

3.4 OFFSITE IMPROVEMENTS ANALYSIS

This section of the drainage study will address the area to the south and southwest of the Newland Sierra project frontage, as the project site itself has been studied in the earlier sections of this study. Information on the existing storm drain systems for Deer Springs Road and Twin Oaks Valley Road were obtained from as-built drawings and field investigation.

Existing Conditions

Basins 1-6000 encompass the proposed road widening from the easterly edge of the Merriam Mountains frontage to the end of the road widening to the south of Cassou Road. Please refer to the Off Site Improvements Drainage Exhibit in Appendix C for geographic information on the existing conditions basins.

Runoff from Basin 1 flows southeast through mixed agricultural and residential land uses, consisting of 203 acres. Drainage from this basin crosses Twin Oaks Valley Road at approximately 1600 feet south of Cassou Road. The existing storm drain system at this crossing consists of a rectangular double box culvert (3'x6') and drains to an unlined open channel on the east side of Twin Oaks Valley Road. The open channel drains into a flood control basin.

Basin 1 Inlet – West Side Twin Oaks Valley Road


Basin 1 Outlet- East Side Twin Oaks Valley Road


Basin 1 Outlet – Downstream Open Channel

In the existing condition, Basin 6000 consists of approximately 133 acres of mixed agricultural and residential land uses. A sparsely developed ridgeline forms the upper edge of Basin 6000. Runoff drains to the southeast along two major streamlines, confluencing in the lower third of the basin. From here, the runoff continues to the southeast and to the intersection of Twin Oaks Valley Road and Cassou Road. The runoff is collected by CMP risers to the north and south of Cassou Road and conveyed southeasterly through the Twin Oaks Elementary School first in a 42" HDPE pipe and then a 48" HDPE storm drain. This storm drain outfalls to an unlined open channel at the southeast corner of Twin Oaks Elementary School.

Basin 6000 Inlet – NW Corner Cassou Road

The existing Basin 2000 is made up of primarily agricultural land uses, and contains 57 acres. Runoff again flows from a high point in the northwest towards the southeast, crossing Twin Oaks Valley Road at the intersection of Buena Creek Road. On the north side of Buena Creek Road, the runoff is collected in an 18" CMP culvert, while on the south side it is collected in three 18" RCP Pipes. On the east side of Twin Oaks Valley Road, the culverts discharge to an unlined open channel.

Basin 2000 Inlet – SW Corner Buena Creek Road

Basin 2000 Outlet – Downstream open channel

Basin 4000 consists of 1.4 acres of a superelevated portion of Twin Oaks Valley Road, between Sycamore Drive and Equestrian Court. Runoff is conveyed in gutters on Twin Oaks Valley Road and heads east down Equestrian Court towards a concrete swale and channel.

Basin 4000- Drainage along the easterly curb of Twin Oaks Valley Road heads east down Equestrian Court

Basin 4000 – Downstream to concrete swale/open channel

Basin 5000 consists of 0.9 acres of the eastern half of a crowned portion of Twin Oaks Valley Road, between Equestrian Court and Olive Street. Runoff is conveyed by curb and gutter and turns the corner at Olive Street, heading east.

Basin 5000- Drainage from the easterly half of Twin Oaks Valley Road heads east down Olive Street.

Basin 6000 consists of 0.8 acres of the eastern half of a crowned portion of Twin Oaks Valley Road, between Olive Street and Cassou Road. Runoff is guided by an AC berm for part of the way, and then runs off into a ditch in an open field alongside Twin Oaks Valley Road that heads east along Cassou Road.

Basin 6000 – Outlet runs down Cassou to open channel

Proposed Conditions

Deer Springs Road is being widened and re-aligned between Twin Oaks Valley Road and Sarver Lane. Along this "off-site" stretch, storm drain is proposed along with water quality basins to treat the runoff. The sub-basins west of 24.2 and north of Mulberry Drive all drain to a proposed Stevensen Creek flood control system which includes a channel and associated box culverts. This system is analyzed in the Floodplain Revision and No-Rise Report for Stevenson Creek and Twin Oaks Valley Creek. Sub-basins 24.9, 34.9, 35.7, 35.6, 35.5, 35.4, 35.3, 35.2, 35 and 35.0 all have outfalls that drain to this system and are summarized below.

Stevenson Creek channel and box culvert outfalls

	Post-Developed (Proposed) (Includes Detention Where Present))				
Sub-basin	Area (acre)	Length (ft)	Q_{out} (cfs)	Tc (min)	Note
24.9	553.0	10962	634	22.6	Entrance to Sarver Lane box culvert
34.9	59.2	4195	135	15.0	SD confluences with Sarver Lane box culvert
35.7	4.0	2135	27	6.3	WQ basin outfall to channel
35.6	0.4	535	3.5	3.8	WQ basin outfall to channel
35.5	0.5	460	3.2	4.2	WQ basin outfall to channel
35.4	0.8	710	5.5	5.5	WQ basin outfall to channel
35.3	1.0	655	8.6	5.1	WQ basin outfall to channel
35.2	0.5	503	3.8	4.0	WQ basin outfall to channel
35.1	1.1	785	7.5	6.4	WQ basin outfall to channel
35.0	0.6	555	4.2	5.9	WQ basin outfall to channel

The sub-basins south of Mulberry Drive drain to a variety of outfalls. Sub-basin 4000 drains easterly down Equestrian Court. Sub-basin 2000 drains to a small natural channel that leads east. Sub-basin 5000 drains easterly down Olive Street. Sub-basin 6000 leads to an existing storm drain in Cassou Road. Results from these sub-basins can be found in the table below.

Twin Oaks Valley Road sub-basins

	Pre-Developed (Existing)				Post-Developed (Proposed) (Includes Detention Where Present))				Flow Change
Sub-basin	Area (acre)	Length (ft)	Q (cfs)	Tc (min)	Area (acre)	Length (ft)	Q_{out} (cfs)	Tc (min)	ΔQ (cfs)
4000*	1.4	1460	4	20.1	0.5	275	4	3.9	0
5000*	0.9	1080	4	10.3	1.5	1040	8	9.8	+4
2000	57.0	3490	64	24.3	58.7	3576	71	24.4	+7
6000	133	5090	145	26.8	133.4	5094	150	26.8	+5
1	203	6070	240	28.3	203.2	6100	245	28.2	+5

While Basins 1, 6000, 2000 and 5000 exhibit slight increases in overall discharge, the level of significance of the increase will not be large enough to adversely impact the downstream channels and drainage facilities.

4. SUMMARY

The Newland Sierra project is a large-scale master planned community with multiple neighborhoods proposed for single family and multi-family living. It will contain access drives and a relatively small area for commercial development. Of the 1983 acres of property, over two thirds of the project is proposed as un-graded terrain preserving its natural resources such as wetlands and steep slopes.

Hydrology calculations have been run on the existing conditions of the site to quantify natural flows. Subbasins were derived from known downstream points of concentration, such as pipes, culverts or major confluences. Once flows were quantified, downstream conditions were analyzed to check for capacity.

Runoff coefficients (C) for the rational method analysis were determined for each subarea based upon the Hydrologic Soil Group (HSG) of the underlying soil as well as the land use pertaining to the conditions under consideration.

In the post-developed conditions, it was of specific interest to eliminate any diversion of drainage areas and to minimize or improve any downstream impacts. Through grading and storm drain systems, diversion is less than a tenth of an acre in all the major basins. In situations where there was an increase of storm runoff, detention basins are proposed to reduce peak flows and serve as water quality mitigation.

This study addresses hydrologic impacts after developed conditions have been reached. By protecting the drainage basin areas, using storm drains and implementing detention basins where required, there will be minor impacts downstream. Many cases show there are no changes to the existing watersheds, and in some cases impacts are actually reduced. Downstream conveyances have been analyzed and proven adequate in most cases; where necessary, mitigation or improvement measures are proposed.

Drainage Mitigation Measures

In addition to utilizing existing detention storage, the mitigation measures summarized in Table 4.1 are proposed:

Table 4.1 Summary of Drainage Mitigation Measures

Subbasin / Location	Proposed Mitigation Measure
Subbasin 13	Construct detention/water quality basins as part of grading for that area
Subbasin 16	Construct a detention/water quality basin as part of grading for that area
Subbasin 19	Construct a detention/water quality basin as part of grading for that area
Subbasin 20	Construct a detention/water quality basin as part of grading for that area
Subbasin 21	Construct a pair of detention/water quality basins as part of grading for that area
Subbasin 22	Construct a detention/water quality basin as part of grading for that area
Subbasin 23	Construct a detention/water quality basin as part of grading for that area
Subbasin 23.4	Construct a detention/water quality basin as part of grading for that area
Subbasin 24	Construct a detention/water quality basin as part of grading for that area
Subbasin 25	Construct three detention/water quality basins as part of grading for those areas
Subbasin 26	Construct a detention/water quality basin as part of grading for that area
Subbasin 27	Construct a detention/water quality basin as part of grading for that area
Subbasin 28	Construct a detention/water quality basin as part of grading for that area
Subbasin 29	Construct a detention/water quality basin as part of grading for that area

5. METHODOLOGY & MODEL DEVELOPMENT

5.1 COUNTY OF SAN DIEGO DRAINAGE DESIGN CRITERIA

Design Frequency – the flood frequency for determining the design storm discharge is 50 years for drainage that is upstream of any major roadway and 100 years frequency for all design storms at a major roadway, crossing the major roadway thereafter. The 50-year storm shall be contained within the pipe and not encroach into the travel lane. For the 100-year storm this includes allowing one lane of a four-lane road (four or more lanes) to be used for conveyance without encroaching onto private property outside the dedicated street right-of-way. Natural channels that remain natural within private property are excluded from the right-of-way guideline.

Design Method – The choice of method to determine flows (discharge) shall be based on the size of the watershed area. For an area 0 to approximately 1 square mile the Rational Method or the Modified Rational Method shall be used. For watershed areas larger than 1 square mile the NRCS hydrologic method shall be used. Please check with the governing agency for any variations to these guidelines.

5.2 RATIONAL METHOD HYDROLOGIC ANALYSIS

Computer Software Package – AES-2014
Design Storm – 100-year return interval
Land Use – Natural, Residential and Commercial Development

Soil Type – . Hydrologic Soil Groups for each subbasin were taken from the San Diego County Soils Interpretation Study: Hydrologic Soil Groups – Runoff Potential map (Appendix E2).

Runoff Coefficient – Runoff coefficients were in accordance with the San Diego County Hydrology Manual 2003, based on a conservative imperviousness assumption. See Section 3.1.1 for details.

Method of Analysis – The Rational Method is the most widely used hydrologic model for estimating peak runoff rates. Applied to small urban and semi-urban areas with drainage areas less than 0.5 square miles, the Rational Method relates storm rainfall intensity, a runoff coefficient, and drainage area to peak runoff rate. This relationship is expressed by the equation:

$Q =$ CIA, where:

$Q =$ The peak runoff rate in cubic feet per second at the point of analysis.

C = A runoff coefficient representing the area – averaged ratio of runoff to rainfall intensity.

I = The time-averaged rainfall intensity in inches per hour corresponding to the time of concentration.

A = The drainage basin area in acres.

To perform a node-link study, the total watershed area is divided into sub-areas which discharge at designated nodes.

The procedure for the sub-area summation model is as follows:

(1) Subdivide the watershed into an initial sub-area and subsequent sub-areas. Per the 2003 San Diego County Hydrology Manual, flow-line for initial sub-area ranges from 50 ft to 100 ft long; initial sub-areas generally are less than 1 acre in size.

(2) Assign upstream and downstream node numbers to each sub-area.

(3) Estimate an initial T_c by using the appropriate nomograph or overland flow velocity estimation.

(4) Using the initial T_c, determine the corresponding values of I. Then Q = CIA.

(5) Using Q, estimate the travel time between this node and the next by Manning's equation as applied to particular channel or conduit linking the two nodes. Then, repeat the calculation for Q based on the revised intensity (which is a function of the revised time of concentration)

The nodes are joined together by links, which may be street gutter flows, drainage swales, drainage ditches, pipe flow, or various channel flows. The AES-2014 computer sub-area menu is as follows:

SUBAREA HYDROLOGIC PROCESS

1 Confluence analysis at node
2 Initial sub-area analysis (including time of concentration calculation)
3 Pipeflow travel time (computer estimated)
4 Pipeflow travel time (user specified)
5 Trapezoidal channel travel time
6 Street flow analysis through subarea
7 User-specified information at node
8 Addition of subarea runoff to main line
9 V-gutter flow through area
10 Copy main stream data to memory bank
11 Confluence main stream data with a memory bank
12 Clear a memory bank

At the confluence point of two or more basins, the following procedure is used to combine peak flow rates to account for differences in basin times of concentration. This adjustment is based on the assumption that each hydrograph is triangular.

(1). If the collection streams have the same times of concentration, then the Q values are directly summed,

$$Q_p = Q_a + Q_b;\ T_p = T_a = T_b$$

(2). If the collection streams have different times of concentration, the smaller of the tributary Q values may be adjusted as follows:

(i). The most frequent case is where the collection stream with the longer time of concentration has the larger Q. The smaller Q value is adjusted by a ratio of rainfall intensities.

$$Q_p = Q_b + Q_a\,(I_b/I_a);\ T_p = T_a$$

(ii). In some cases, the collection stream with the shorter time of concentration has the larger Q. Then the smaller Q is adjusted by a ratio of the T values.

$$Q_p = Q_b + Q_a\,(T_b/T_a);\ T_p = T_b$$

5.3 DETENTION BASIN ANALYSIS

Analysis of the proposed detention basins was performed using the HEC-HMS software program. Critical inputs for the software are described below. Screen captures, results, and electronic files from the program can be found in Appendix C.

Computer Software Package – HEC-HMS Version 4.0 by US Army Corps of Engineers Hydrologic Engineering Center

Basin Inflow: Discharge Gage
Using the peak runoff, time of concentration, area, and average runoff coefficient at the detention basin's hydrology node, an inflow hydrograph to the detention basin was developed using the RatHydro program. The data from the hydrograph was input into HEC-HMS as a discharge gage using manual entry. The time of concentration was used as the time interval. In some cases, the preset time intervals in HEC-HMS did not match the time of concentration, so the RatHydro values were interpolated as needed using the highest common denominator allowed by HEC-HMS. In some cases, an inflow hydrograph time interval of one minute was necessary. The values generated by the RatHydro program and the time interval adjustments (if needed) are included in Appendix C for reference.

Reservoir:
A reservoir was used to model the detention basin. The reservoir was modeled using the outflow structures method and the elevation-storage method. For detention basins functioning purely for peak detention, the reservoir was assumed to be empty at the start of the storm. For detention basins which also function for pollutant control and hydromodification purposes, the "dead" surface ponding volume below the detention basin outlet orifice was ignored, and the bottom of the detention basin was assumed to start above the "dead" storage volume. An appropriate tailwater was assigned to each basin. For the detention basins within the project, a tailwater elevation equal to the outlet orifice diameter is assumed to reflect the presence of the low flows from the subdrain on the pollutant control/hydromodification portion of the basin.

Outlet:
For the proposed detention basins, the outlets are modeled as orifices. Conceptually, the detention basins outlets will consist of a Type G catch basin with an orifice opening in the side. The top opening of the Type G catch basin will serve as emergency overflow for the basin. The orifice diameter was selected to yield post-detention flow rates that are less than the existing peak flow. In cases where the proposed detention basin also functions as a water quality/hydromodification IMP, the outlet orifice is raised 1' above the bottom of the basin and the volume below the outlet orifice is ignored. An orifice coefficient of 0.60 was selected for a sharp, clean

edge per table 6-2 of the County of San Diego Hydraulic Design Manual. Details of the proposed outlet structures are included in the exhibits in Appendix C.

Control Specifications:
The control specifications were set to match the duration of the inflow hydrograph. The time interval in the control specifications was set to one minute to capture all possible peak flows.

Elevation-Storage Function:
The elevation-storage data was input as paired data, and was derived from the contours of the detention basin. Exhibits showing the existing and proposed contours as well as tables showing the elevations, depths, contour area, and volume of the detention basins can be found in Appendix B.

Results of the HEC-HMS modeling can be found in Appendix B, in the form of a summary table and time series data showing the inflow, storage, water surface elevation, and outflow.

6. REFERENCES

Hydraulic Design Manual, County of San Diego: September 2014

Hydrology Manual, San Diego County: June 2003

Caltrans Drainage Design, 11-SD-15, EA 095041: 1974
(Interstate 15, Stations 1765 to 1845+75)

Caltrans Drainage Design, 11-SD-15, EA 144811: 1977
(Interstate 15, Stations 1833 to 1918)

Caltrans As-Built, Contract 11-095044: November 1979

Caltrans As-Built, Contract 11-095054: May 1980

County of San Diego, Road Survey No. 521 (Twin Oaks Valley Road): Oct 1959

County of San Diego, Road Survey No. 1040 (Deer Springs Road): Nov 1959

County of San Diego As-Built, Street Improvement Plan for Portion of Deer Springs Road and Sarver Lane (CG 3852): December 1993

Advanced Engineering Systems (AES) 2012, Hydrology Software

Haestad FlowMaster, Hydraulics Software

Hydrologic Modeling System HEC-HMS User's Manual Version 4.0, US Army Corps of Engineers: December 2013

HEC-RAS River Analysis System Hydraulic Reference Manual Version 4.1, US Army Corps of Engineers: January 2010

Flood Insurance Study San Diego County, California and Incorporated Areas, Federal Emergency Management Agency: May 2012

SECTION 7

DECLARATION OF RESPONSIBLE CHARGE

NEWLAND SIERRA
DECLARATION OF RESPONSIBLE CHARGE

I HEREBY DECLARE THAT I AM THE ENGINEER OF WORK FOR THIS PROJECT, THAT I HAVE EXERCISED RESPONSIBLE CHARGE OVER THE DESIGN OF THE PROJECT AS DEFINED IN SECTION 6703 OF THE BUSINESS AND PROFESSIONS CODE AND THAT THE DESIGN IS CONSISTENT WITH CURRENT STANDARDS. I UNDERSTAND THAT THE CHECK OF THE PROJECT DRAWINGS AND SPECIFICATIONS BY THE COUNTY OF SAN DIEGO IS CONFINED TO A REVIEW ONLY AND DOES NOT RELIEVE ME, AS ENGINEER OF WORK, OF MY RESPONSIBILITIES FOR THE PROJECT DESIGN.

FUSCOE ENGINEERING
6390 GREENWICH DRIVE SUITE 170
SAN DIEGO, CA 92122

SIGNED: ______________________ *DATE:* 2/24/17

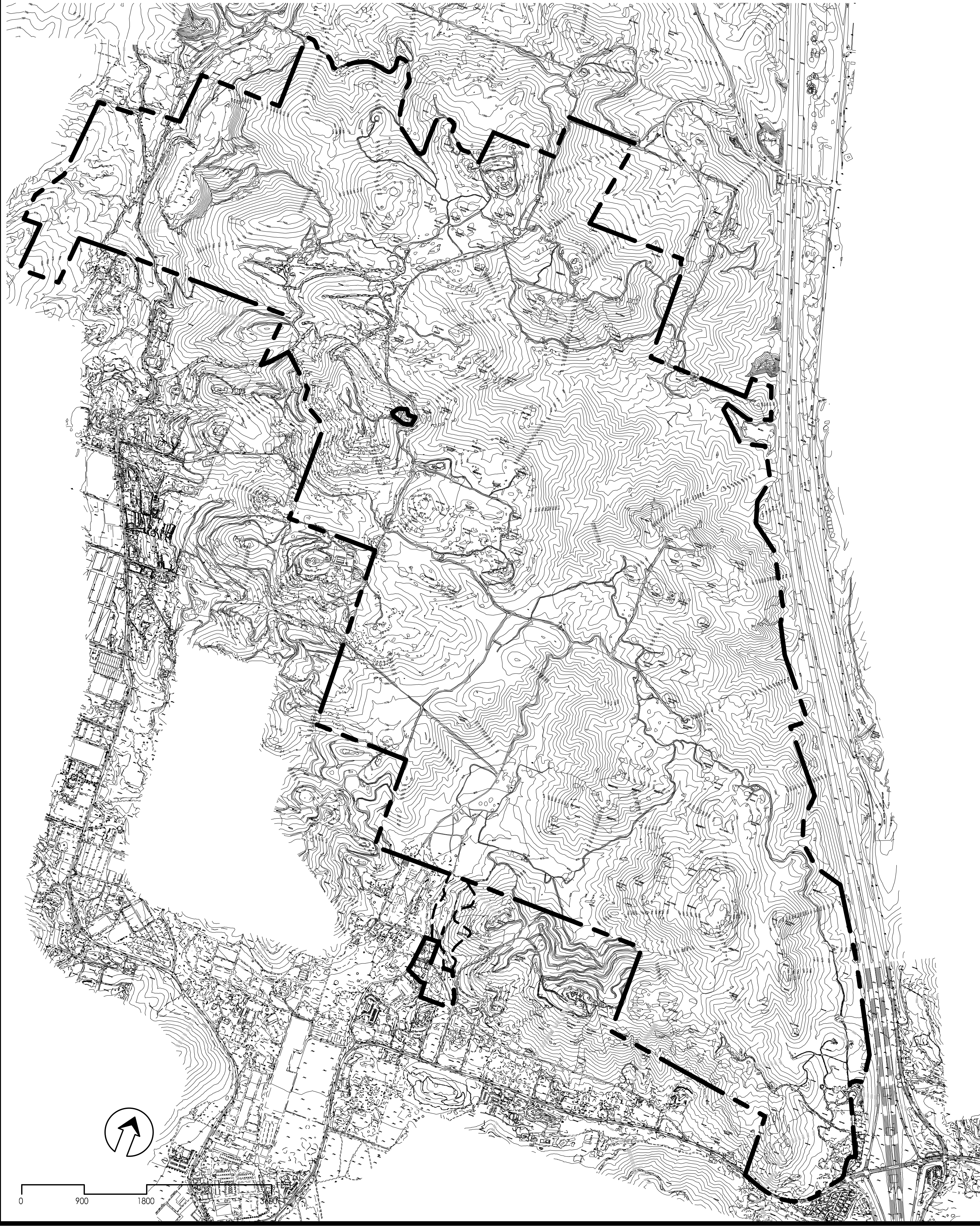
KENNETH T. KOZLIK
R.C.E. #71883 EXP 12/31/17

REGISTERED PROFESSIONAL ENGINEER
KENNETH T. KOZLIK
C 71883
EXP. 12/31/17
CIVIL
STATE OF CALIFORNIA

SECTION 8

PROJECT TOPOGRAPHIC MAP

NEWLAND SIERRA
TOPOGRAPHY



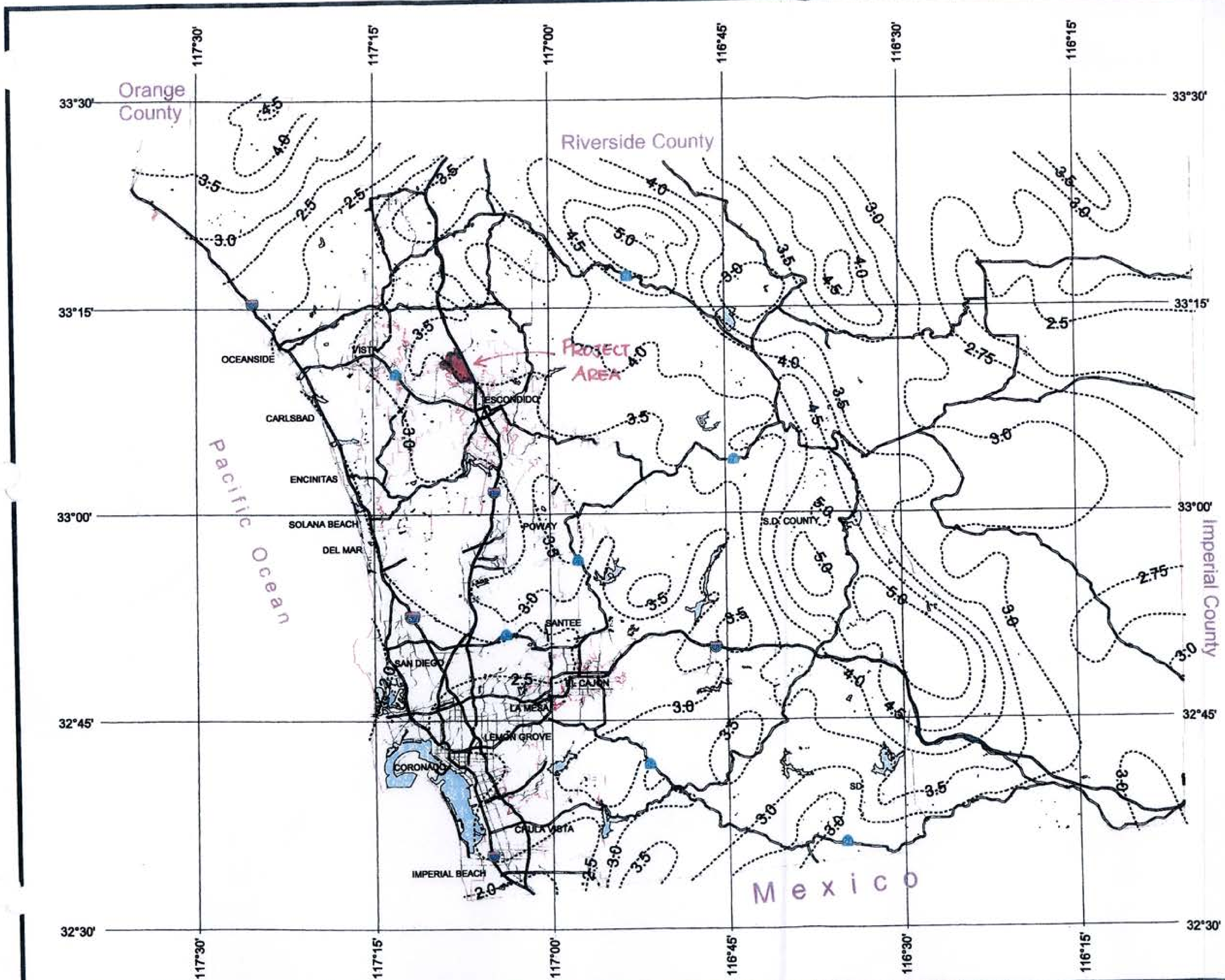
M:\PROJECTS\2660 NEWLAND COMMUNITIES\002-02 NEWLAND SIERRA\ENG\STORMWATER\DRAINAGE STUDY\MAIN REPORT\08 TOPOGRAPHY EXHIBIT.DWG (01 PLOTTED BY: kkozlik

FIGURE 9

COUNTY OF SAN DIEGO
RAINFALL ISOPLUVIALS MAPS

100-YEAR RAINFALL EVENT

6 HOURS
24 HOURS



County of San Diego Hydrology Manual

Rainfall Isopluvials

100 Year Rainfall Event - 6 Hours

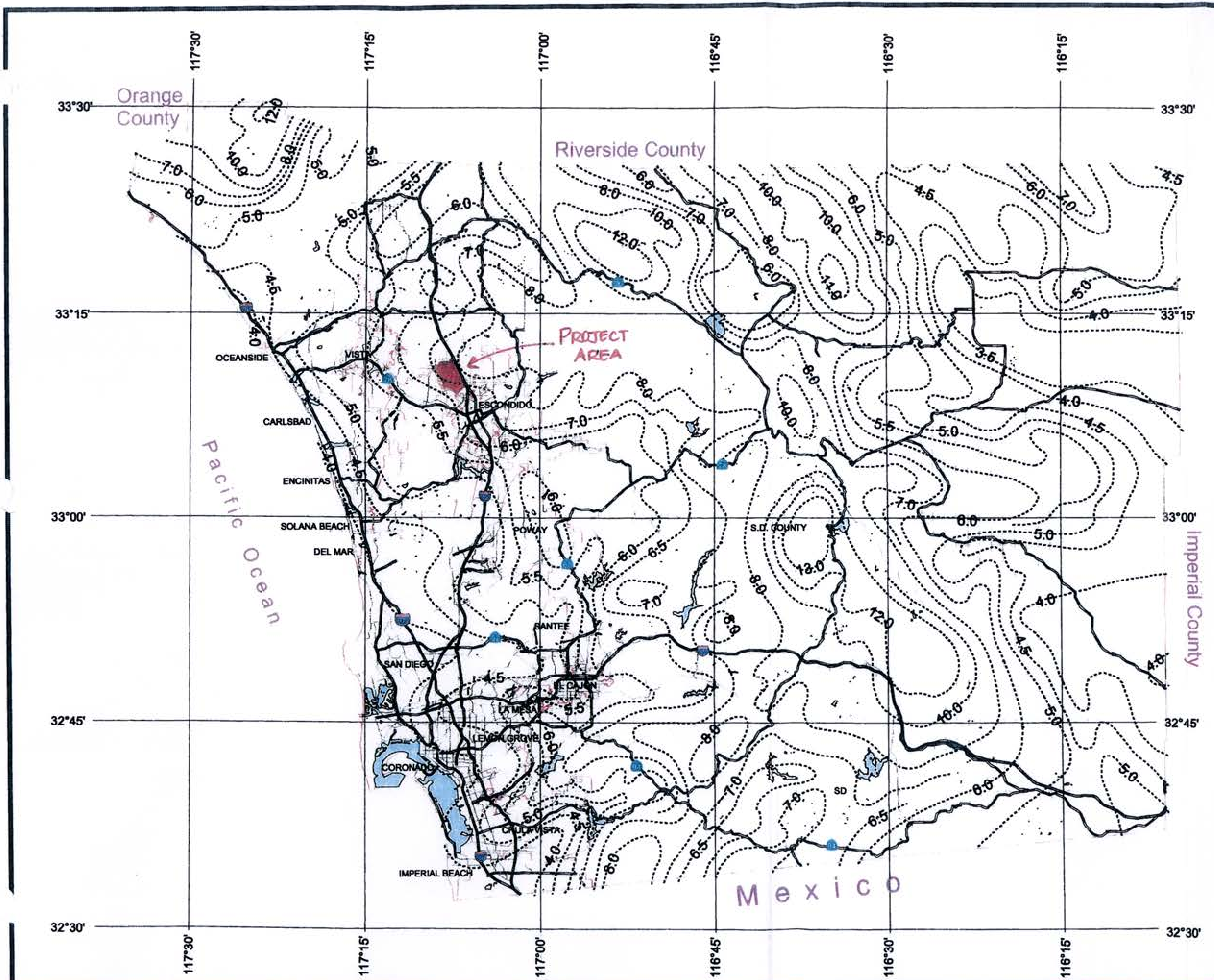
-------- Isopluvial (inches)

DPW GIS
Department of Public Works
Geographic Information Services

SanGIS
We Have San Diego Covered!

N E S W

3 0 3 Miles



County of San Diego Hydrology Manual

Rainfall Isopluvials

100 Year Rainfall Event - 24 Hours

-------- Isopluvial (inches)

DPW GIS

Department of Public Works

SanGIS

We Have San Diego Covered!

N W E S

3 0 3 Miles

SECTION 10

WATERSHED GEOMETRIC INFORMATION MAP

FOR
BASIN A
BASIN B
BASIN C
BASIN D
BASIN E

EXISTING CONDITIONS

PLEASE REFER TO APPENDIX E IN VOLUME 4 FOR MAPS

SECTION 11

WATERSHED GEOMETRIC INFORMATION MAP

FOR
BASIN A
BASIN B
BASIN C
BASIN D
BASIN E

POST-DEVELOPED CONDITIONS

PLEASE REFER TO APPENDIX E IN VOLUME 4 FOR MAPS