6.0 MODEL RESULTS

6.1 PURPOSE

The purpose of this chapter is to present the results of each model simulation scenario. The results of each scenario are evaluated based on LOS performance criteria. Development scenarios and LOS criteria are described in **Chapter 5.0**.

This chapter is divided into the following sections:

- Existing Level of Development Results
  - Scenario 1 – Existing Dry Weather Flow
  - Scenario 2 – Peak Wet Weather Flow
- Future Development Results
  - Scenario 3 – Buildout of Existing Sewer-sheds
  - Scenario 4 – Buildout of City Limits
  - Scenario 5 – Buildout of the SOI
  - Scenario 6 – Buildout of the SOI, plus Regional Flows
- Pump Station Capacity Evaluation

6.2 EXISTING LEVEL OF DEVELOPMENT RESULTS

The hydraulic model of the existing collection system was assessed to determine the capacity of the existing trunk network, identify hydraulic deficiencies, and to determine improvements to the sewer system, if necessary to accommodate flow under ADWF and PWWF conditions.

6.2.1 Scenario 1 – Existing Dry Weather Flow

The existing system was modeled and calibrated under "dry weather" conditions, the results of this model were used to evaluate the LOS of sewers within the collection system under typical conditions. It should be noted that the dry weather flow monitoring period from which the existing system was modeled, was during a time of high ground water infiltration and modeled DWF is higher than ADWFs observed at the WWTRF during the dry season (July – September).

There is no surcharging predicted to occur in the existing system under dry weather conditions. A peak dry weather flow of 10.2 MGD was predicted to occur at the WWTRF. The existing modeled system maintains HLRs and d/D ratios below 50 percent under peak dry weather flow conditions. The maximum velocity is approximately 7 fps for gravity mains. There are no LOS failures predicted under these conditions.

Level of service criteria results of the existing system under dry weather flow conditions are presented in **Figure D-1** through **Figure D-6** in **Appendix D.**

6.2.2 Scenario 2 – Existing Wet Weather Flow

A 10-year, 24-hour design storm was applied to the calibrated wet-weather flow model. The results of this simulation were used to evaluate the existing system under wet weather flow conditions. Modeled LOS results are presented in **Appendix D**. **Figure D-7** shows peak modeled flows in the existing system under design storm conditions. **Figure D-8** shows the corresponding HLR and **Figure D-9** depicts the residual capacity. The minimum freeboard expressed as depth below manhole rim, is shown on **Figure D-10**. The maximum simulated velocity is shown on **Figure D-11**. **Table 6-1** shows the summary of simulated flows for each of the flow monitoring locations under design storm conditions.

Table 6-1 Flow Characteristics of the Existing System under Existing Conditions (1)

Sewer-shed	Catchment Area (Acres)	Modeled ADWF (MGD)	Modeled Peak ADWF (MGD)	PWWF, 10-year, 24-hour Rainfall (MGD)	Modeled Peak RDII Flow (MGD)	Modeled Peak RDII Rate (gpd/acre)
Site 1	495 (2)	0.68	1.63	3.36	1.73	3,485
Site 2	297	0.12	0.22	0.84	0.63	2,107
Site 3	291	0.44	0.66	5.18	4.52	15,520
Site 4	2,000 (3)	1.80	3.77	14.82	11.06	5,529
Site 5	791	0.48	0.20	6.19	6.00	7,581
Site 6	134	0.25	0.32	1.88	1.56	11,587
Site 7	337	0.39	0.16	5.51	5.35	15,875
Site 7A	242	0.18	0.38	0.97	0.60	2,460
Site 8	1,114	0.48	0.84	1.99	1.15	1,028
Site 9	411	0.84	1.03	3.27	2.24	5,447
Site 10	724	0.54	0.97	3.15	2.18	3,010
Total:	**6,837**	**6.18**	**10.18**	**47.17**	**36.99**	**5,410**

(1) GWI occurred throughout the flow monitoring study. Without a measurement of baseline ADWF, GWI could not be determined for each sewer-shed. Empirical estimates of GWI within each basin were considered but excluded from this analysis due to inaccuracy (The remaining estimate of ADWF was much higher than what is observed at the WWTRF when flow recedes to baseline levels). Based on WWTRF flow data, approximately 2.0 MGD of GWI occurred during the flow monitoring period.

(2) 549 acres of land surrounding the Airport is considered open space and does not contribute to RDII. Therefore, were exclude from the catchment area simulated in the hydraulic model.

(3) The existing SMD1 sewer-shed (Shed 4) is approximately 3,769 gross acres, for calibration purposes a value of 2,000 net acres was approximated as contributing to RDII.

As shown in **Table 6-1**, the SMD1 sewer-shed (Site 4) has a high peak RDII flow (~ 11 MGD), meaning it contributes a large portion of the WWTRF's total I/I flow (~ 37 MGD). Sewer-sheds 3, 6, and 7 have high RDII rates, meaning they contribute a large quantity of I/I per acre. The RDII rate can be used to provide a normalized comparison of I/I between sewer-sheds.

Under existing conditions, the design storm is predicted to generate a PWWF of 47.2 MGD at the WWTRF. This storm event is predicted to cause surcharging in several reaches of the existing collection system. To help identify the extent of the predicted surcharging, hydraulic grade line (HGL) profiles have been included in **Appendix C** for areas of concern. It should be noted that the profiles also include the results of other growth scenarios, to be discussed in the following sections. **Figure C-1** shows the profile locations in plan view.

The following capacity constraints within the existing system have been identified from the PWWF simulation:

HGL Profile 1: Old Town North Trunk – Part A (Figure C-2)
Location: 3rd Street, O Street, and 4th Street
Surcharged Manholes: NW457SS04 to NW422SS55
Proposed Improvements: CIP 1
Problem Description: The modeled sewers upstream of the intersection of 5th Street and Joiner Parkway show a capacity constraint under PWWF conditions. In this reach of the system, pipe size and slope vary, resulting in varied capacity in each sewer segment and non-ideal flow conditions. Surcharging is predicted to occur, and many manholes along this reach have less than 8-feet of depth between the manhole rim and pipe crown elevations (available freeboard). Therefore, no surcharging is allowed under LOS criteria. An SSO occurs downstream of this profile, and surcharge depth is predicted to reach 3.7 feet above the pipe crown elevation in manhole NW422SS57 (freeboard = 2.9 feet).

HGL Profile 2: Old Town North Trunk – Part B (Figure C-3)
Location: Q Street and 5th Street
Surcharged Manholes: NW422SS55 to NW388SS015
SSO Manholes: NW422SS53 and NW456SS03 (excluded from profile upstream of NW457SS01 on 4th Street)
Proposed Improvements: CIP 1
Problem Description: This profile presents the remaining modeled sewers upstream of the intersection of 5th Street and Joiner Parkway, along the Old Town North Trunk. Like those upstream, pipe size and slope are inconsistent in this reach of the system and manholes have less than 8-feet of available freeboard. An SSO is predicted to occur in manhole NW422SS53, near the intersection of Q Street and Cheryl Court. Surcharging in manhole NW422SS29 is predicted to reach 4.9 feet above the pipe crown (freeboard = 0.45 feet).

HGL Profile 3: Old Town South Trunk – Part A (Figure C-4)
Location: Joiner Parkway and 2nd Street
Surcharged Manholes: NW457SS12 to NW389SS39
Proposed Improvements: CIP 2
Problem Description: The sewers along the Old Town South Trunk, in 2nd Street between Joiner Parkway and J Street are predicted to cause a capacity constraint, similar to the sewers in 5th Street. No SSOs are predicted to occur in this reach of the system. Manholes between NW423SS35 and NW389SS039 have less than 8-feet of available freeboard, therefore no surcharging is allowed under LOS criteria. Surcharge depth is predicted to reach 3.4 feet above the pipe crown in manhole NW457SS12 (freeboard = 9.8 feet). The minimum freeboard along this profile is predicted as 5.6 feet below the rim of manhole NW389SS038.

HGL Profile 4: Aviation Trunk – Part A (Figure C-5)
Location: Aviation Boulevard (Near the Airport)
Surcharged Manholes: NW282SS01 to NW283SS05
Proposed Improvements: CIP 5
Problem Description: Surcharging is predicted to occur upstream of the Nicolaus Road Pump Station (NRPS) in Aviation Boulevard. This capacity constraint is the result of a reduction in pipe size at manhole NW283SS03, near Venture Drive. The 12-inch portion of the sewer is reduced to 10-inches at this location. Surcharge depth reaches 3-feet above the pipe crown near the pipe size reduction. No SSOs are predicted to occur along this profile in this scenario. The minimum freeboard along this profile is 5.9 feet below the rim of manhole NW282SS01.

HGL Profile 5: Aviation Trunk – Part B (Figure C-6)
Location: Nicolaus Road and Aviation Boulevard
Surcharged Manholes: SE503SS05 to NW319SS01
Proposed Improvements: CIP 5
Problem Description: The 10-inch sewer in Aviation Boulevard continues east down Nicolaus Road to the NRPS. Surcharging occurs along this profile due to insufficient capacity of the 10-inch sewer. Surcharge depth reaches 2.8 feet above the pipe crown in manhole NW283SS05. Manholes NW319SS02, NW319SS03, and NW319SS04 are predicted to surcharge less than a foot with more than 8-feet of available freeboard, meeting LOS criteria. Freeboard remains greater than 8 feet along this profile, under these conditions.

HGL Profile 6: East Lincoln Parkway Pump Station Influent Sewers (Figure C-7)
Location: East Joiner Parkway, upstream of the East Lincoln Parkway Pump Station (ELPPS)
Surcharged Manholes: SE503SS05 to SE502SS11
SSO Manholes: SE502SS13
Proposed Improvements: CIP 3
Problem Description: Surcharging is predicted to occur in sewers immediately upstream of the ELPPS. An SSO is predicted to occur at manhole SE502SS13. Surcharging and SSOs occur when PWWF to the pump station exceeds the reliable pumping capacity. Surcharging in this reach of the collection system is not a result of insufficient pipeline capacity. Further discussion of pump station capacity constraints is presented in **Section 6.4**. Surcharging exceeds 10-feet in this portion of the collection system under wet weather conditions.

HGL Profile 7: North E Street Trunk Sewer (Figure C-8)
Location: 12th Street, downstream of McCourtney Road
Surcharged Manholes: NE521SS25 to NE521SS52
Proposed Improvements: CIP 4
Problem Description: Surcharging is predicted to occur in the 12-inch sewer due to insufficient capacity in the 12-inch portion of this trunk. SSOs are not predicted to occur in this reach of the system under existing PWWFs. Surcharge depth reaches 4.1 feet in manhole NE521SS25. The minimum freeboard along this profile reaches 6.0 feet below the ground surface at manhole NE521SS52.

Sewers exceeding LOS surcharge criteria, within the existing system, under PWWF conditions are presented in **Figure 6-1**.

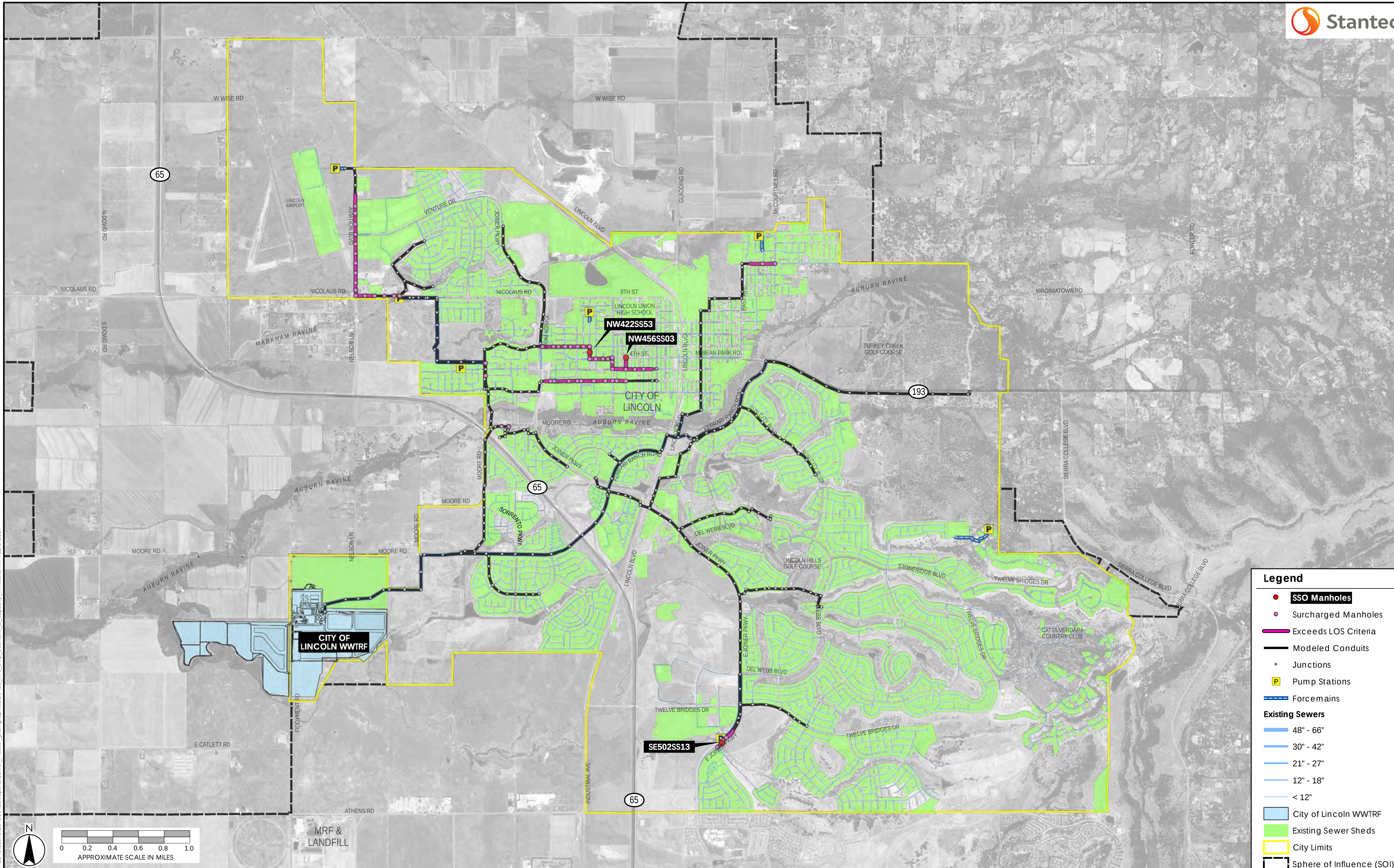

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Figure 6-1
Scenario 2 – Sewers Exceeding LOS Criteria

6.3 FUTURE DEVELOPMENT RESULTS

The existing system model was expanded to include PWWF from future development areas and regional wastewater connections. Future development models were evaluated to assess the impact of additional flow in the existing collection system and to determine the infrastructure needed to serve near-term and long-term development. Plan view figures of LOS criteria results for each modeled scenario are presented in **Appendix D**. Profile views of the future trunk network are available within the City's hydraulic model of the proposed system and are available upon request.

6.3.1 Scenario 3 – Buildout of Existing Sewer-sheds

The wet weather flow model of the existing system was expanded to include flow from future infill development (vacancies within the existing system) within the existing collection system service area. Buildout of vacant parcels added approximately 3.0 MGD to the total ADWF simulated in Scenario 2. A PWWF of approximately 55.0 MGD is predicted to occur at the WWTRF. The addition of flow from infill developments exacerbates capacity restrictions identified in **Section 6.2.2** for Scenario 2. To help identify the extent of pipeline capacity limitations, hydraulic grade line (HGL) profiles have been included in **Appendix C** for areas of concern. It should be noted that the profiles also include the results of other growth scenarios, previously discussed. **Figure C-1** shows the profile locations in plan view.

The following capacity constraints within the existing system have been identified under PWWF conditions and the addition of wastewater from infill developments:

HGL Profile 1: Old Town North Trunk – Part A (Figure C-2)
Location: 3rd Street, O Street, and 4th Street
Surcharged Manholes: NW457SS04 to NW422SS55
Proposed Improvements: CIP 1
<u>Problem Description</u>: Surcharge depth along this reach of the Old Town North Trunk is only slightly elevated when compared to the results of Scenario 2 (Existing PWWF). The downstream SSO impacts this reach and limits the HGL elevation. Minimum freeboard along this profile is 2.9 feet below the rim elevation of manhole NW422SS57.

HGL Profile 2: Old Town North Trunk – Part B (Figure C-3)
Location: Q Street and 5th Street
Surcharged Manholes: NW422SS55 to NW388SS015
SSO Manholes: NW422SS53 and NW456SS03 (excluded from profile upstream of NW457SS01 on 4th Street)
Proposed Improvements: CIP 1
Problem Description: The HGL between manholes NW422SS27 and NW422SS016 is predicted to increase by approximately 0.35 feet when compared to results from Scenario 2. Total overflow volume simulated in the model increases from 0.08 MG to 0.12 MG at manhole NW422SS53, and 0.06 MG to 0.08 MG at manhole NW456SS03, with the addition of flow from infill development. The minimum freeboard along this reach is 0.3 feet below the rim elevation of manhole NW422SS29.

HGL Profile 3: Old Town South Trunk – Part A (Figure C-4)
Location: Joiner Parkway and 2nd Street
Surcharged Manholes: NW457SS16 to NW389SS038
Proposed Improvements: CIP 2
Problem Description: Surcharging along this portion of the Old Town South Trunk is exacerbated by the addition of flow. Manholes NW457SS16, NW457SS001, NW457SS13, and NW389SS37 fail to meet LOS criteria in this scenario. Surcharge depth increases, reaching 3.4 feet above the pipe crown at manhole NW457SS12 (freeboard = 7.6 feet). There is insufficient capacity in the 15-inch sewer to convey PWWF in this scenario. The minimum freeboard along this profile is 4.6 feet below the rim elevation of manhole NW423SS009.

HGL Profile 4: Aviation Trunk – Part A (Figure C-5)
Location: Aviation Boulevard (Near the Airport)
Surcharged Manholes: NW281SS08 to NW283SS05
SSO Manholes: NW281SS08, NW281SS10, NW281SS11, NW281SS12, and NW282SS01
Proposed Improvements: CIP 5
Problem Description: SSOs are predicted to occur upstream of the NRPS in Aviation Boulevard. This is partially due to the limited reliable pumping capacity of the NRPS, which becomes overwhelmed by PWWF in this scenario. Further evaluation of the NRPS capacity is presented in **Section 6.4**. The limited capacity of the 10-inch sewer and the additional flow from infill development around the airport, increase the extent of surcharging and SSOs in this portion of the collection system. A total volume of 1.7 MG was predicted to overflow from this reach of the collection system. The HGL along this portion of the Aviation Trunk increases by an average of 7 feet when compared to the results of Scenario 2. Freeboard along this profile is less than 2.6 feet below the ground surface.

HGL Profile 5: Aviation Trunk – Part B (Figure C-6)
Location: Nicolaus Road and Aviation Boulevard
Surcharged Manholes: SE503SS05 to NW319SS04
Proposed Improvements: CIP 5
<u>Problem Description</u>: The addition of flow from infill development and the limited pumping capacity of the NRPS cause severe surcharging along this reach of the Aviation Trunk sewer. Surcharge depth increases to 14.3 feet above the pipe crown at manhole NW319SS04 (freeboard = 0.3 feet). Wastewater backs up along this profile due to insufficient reliable pumping capacity at the NRPS (PWWF > Reliable Pumping Capacity). Backwater effects cause manholes NW319SS02, NW319SS03, and NW319SS04 to fail LOS criteria.

HGL Profile 6: East Lincoln Parkway Pump Station Influent Sewer (Figure C-7)
Location: East Joiner Parkway, upstream of the East Lincoln Parkway Pump Station (ELPPS)
Surcharged Manholes: SE503SS05 to SE502SS11
SSO Manholes: SE502SS13
Proposed Improvements: CIP 3
<u>Problem Description</u>: The HGL within this reach of the collection system increases slightly when compared to the results of Scenario 2. Similar to Profile 1, the SSO at manhole SE502SS13 limits the level of predicted surcharging upstream. The simulated overflow volume increases from 0.015 MG in Scenario 2 to 0.373 MG in Scenario 3. As previously identified, the limited reliable pumping capacity of the ELPPS causes surcharging in this portion of the upstream collection system. No pipeline capacity constraints exist along this profile.

HGL Profile 7: North E Street Trunk Sewer (Figure C-8)
Location: 12th Street, East Avenue, and 9th Street
Surcharged Manholes: NE521SS25 to NE488SS005
Proposed Improvements: CIP 4
<u>Problem Description</u>: Surcharging is predicted to occur between McCourtney Road and D street along the E Street Trunk. The additional flow causes capacity restrictions in the 21-inch sewer in 9th Street and the 18-inch sewer in East Avenue. Higher flow further exacerbates surcharging in the 12-inch portion of the sewer identified as surcharged in Scenario 2. Surcharge depth in the 21-inch sewer is predicted to be less than 1-foot above the pipe crown, and therefore meets LOS criteria (freeboard = 12.8 feet). Surcharging predicted in the 18-inch sewer reaches a depth of 2.6 feet above the pipe crown in manhole NE488SS07 (freeboard = 11.9 feet). Severe surcharging is predicted to occur in the 12-inch sewer, reaching a depth of 12.7 feet above the pipe crown (freeboard = 0.9 feet).

HGL Profile 8: Old Town North Trunk – Part C (Figure C-9)
Location: 5th Street, Chambers Drive, and Douglas Drive
Surcharged Manholes: NW354SS08 to NW355SS27
Proposed Improvements: CIP 6
Problem Description: This profile depicts the confluence of the Old Town North Trunk, Old Town South Trunk, and discharge of the NRPS forcemain located near Thomsen Way. Insufficient capacity in the 30-inch trunk sewer in Chambers Drive causes the manholes between NW388SS01 and NW389SS23 to fail LOS criteria in this scenario. The 30-inch sewer is predicted to experience minor surcharging in Scenario 2, remaining within LOS criteria. Surcharging in the 24-inch sewer upstream of manhole NW388SS01, is the result of backwater effects induced by the downstream capacity limitation. Surcharge depth is predicted to reach 3.5 feet in manhole NW355SS24 (freeboard = 18.6 feet). Minimum freeboard along this profile 10.7 feet below grade.

HGL Profile 9: Old Town South Trunk – Part B (Figure C-10)
Location: Joiner Parkway, 1st Street, and Chambers Drive
Surcharged Manholes: NW390SS21 to NW389SS33
Proposed Improvements: CIP 6
Problem Description: Surcharging is predicted to occur in the 18-inch sewer along the Old Town South Trunk between Joiner Parkway and 3rd Street. Surcharging is the result of insufficient capacity in the 18-inch sewer under PWWF conditions. The manholes between NW389SS37 and NW389SS31 fail to meet LOS criteria, and reach a surcharge depth of 1.5 feet above the pipe crown. Minimum freeboard along this profile remains 10.4 feet below grade.

HGL Profile 10: Lincoln Crossing Trunk (Figure C-11)
Location: Caledon Circle upstream of Ferrari Ranch Road
Surcharged Manholes: SW396SS17 to SW395SS06
Proposed Improvements: CIP 7
Problem Description: Significant surcharging is predicted to occur in the 12-inch sewer serving the Lincoln Crossings subdivision in this scenario. The Lincoln 270 area is assumed to be served through the Lincoln Crossings upon development. The Lincoln 270 area is located near the intersections of Lincoln Boulevard, Highway 65, and 12 Bridges Drive and consists of approximately 100 vacant acres with a mix of public, commercial, and industrial land uses. There is insufficient capacity in the 12-inch sewer to accommodate full buildout of the Lincoln 270 area. Surcharge depth is predicted to reach 14.2 feet above the pipe crown in manhole SW396SS17 (freeboard = 5.8 feet). If the 270 Area is serviced through other means, there is sufficient capacity in the 12-inch sewer to serve the existing sewer-shed.

HGL Profile 11: Nicolaus Road Pump Station Collection Shed – Part A (Figure C-12)
Location: Markham Ravine Sewers
Surcharged Manholes: NW351SS69 to NW352SS30
SSO Manholes: NW317SS11 and NW352SS31
Proposed Improvements: CIP 5
Problem Description: This profile presents sewers upstream of the NRPS along Markham Ravine. These sewers were the subject of the 2015 Flow Monitoring Study performed by V&A. Manhole NW318SS03 was the location of flow monitor 3, and manhole NW318SS01 was the location of flow monitors 4 and 5. The surcharging in this reach is a result of backwater effects from downstream capacity constraints. Surcharging is predicted to occur throughout the NRPS collection shed due to limited reliable pumping capacity. SSOs are predicted to occur at manholes NW317SS11 and NW352SS31. A volume of 0.22 MG is predicted to overflow from these manholes in the simulation. As seen in Figure C-16, this profile remains surcharged after pumping capacity has been increased. The limited capacity of the 15-inch sewers (between manholes NW319SS05 and NW318SS03) connecting this profile to the NRPS, induce backwater effects. The proposed improvements include upsizing these capacity limited sewers.

HGL Profile 12: Nicolaus Road Pump Station Collection Shed – Part B (Figure C-13)
Location: Nicolaus Road East of the NRPS
Surcharged Manholes: NW353SS34 to NW319SS06
Proposed Improvements: CIP 5
Problem Description: Surcharging is predicted to occur in the 18-inch sewer in Nicolaus Road, east of the NRPS. The surcharging is a result of backwater effects caused by insufficient pumping capacity at the NRPS. There are no pipeline capacity constraints associated with this profile, surcharging is due to limited reliable pumping capacity of the NRPS (PWWF > Reliable Pumping Capacity).

Sewers exceeding LOS criteria for surcharging within the existing collection system simulated in Scenario 3 are presented on **Figure 6-2**.

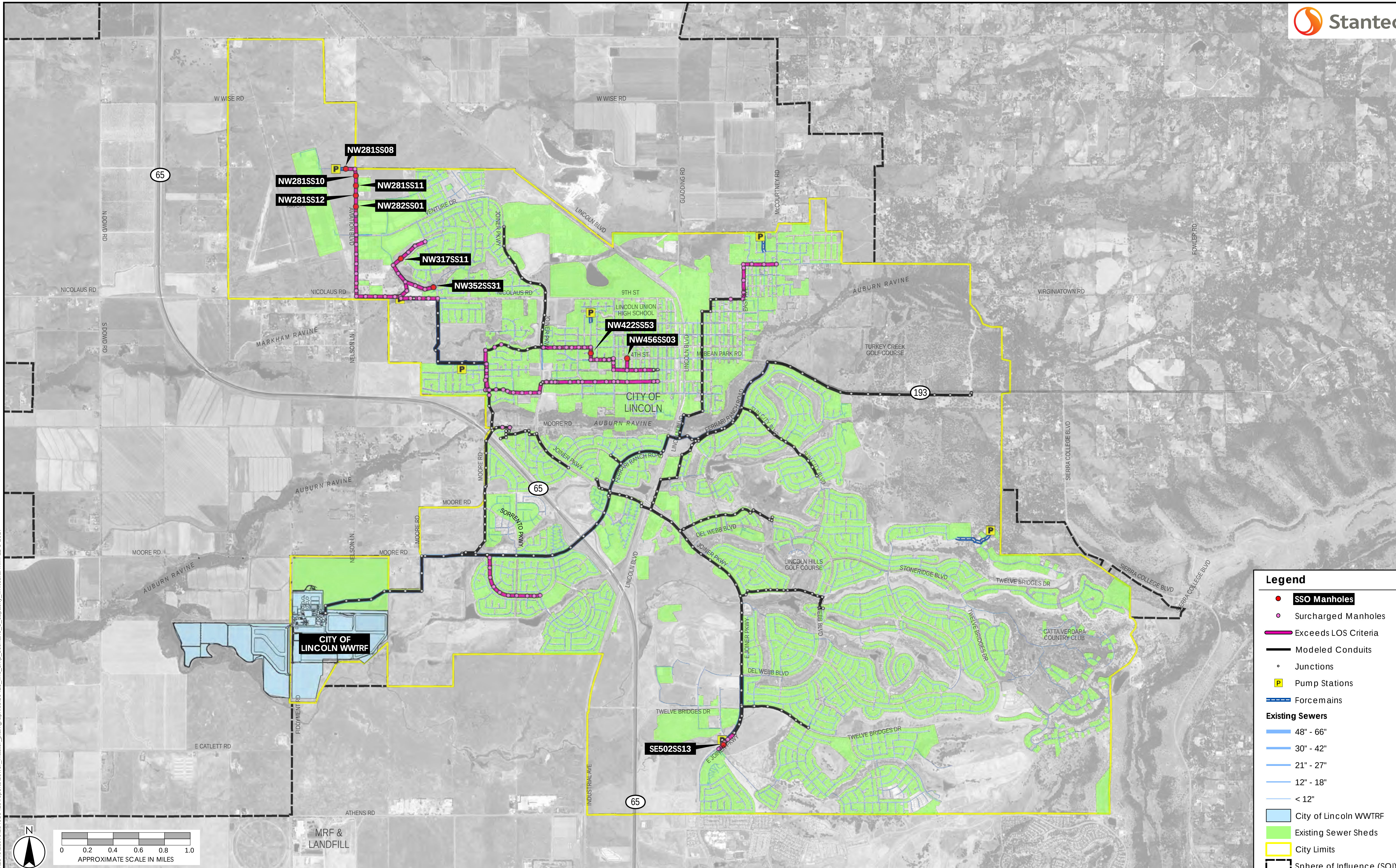

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Figure 6-2
Scenario 3 – Sewers Exceeding LOS Criteria

6.3.2 Scenario 4 – Buildout of City Limits

The existing system model was further expanded to include wastewater flow from Village 1 and Village 7. The addition of flow from these Villages is not predicted to impose any new capacity constraints on the existing system or exacerbate any of those previously identified. A PWWF of 60.3 MGD is predicted to occur at the WWTRF. Flow from Village 1 and Village 7 will be routed through the Regional Sewer Trunk and the Moore Road Trunk. These sewers have enough residual capacity to accept the projected PWWF from these areas. The LOS model results for Scenario 4 are included in **Appendix D**.

Village Specific Plans outline the specific infrastructure required to provide wastewater collection service to these areas. For purposes of this Master Plan, trunk sewers greater than or equal to 12-inches in diameter are considered "infrastructure needed" to provide service to these areas. Inflows were input directly into the model at points where collector sewers less than 12-inches in diameter, are planned to connect to existing trunk sewers. The proposed points of connection for proposed Village 1 and Village 7 sewers are shown on **Figure 6-3**.

Infrastructure Required to Provide Service to Village 1

Three new trunk sewers and an extension of the Regional Sewer trunk will be needed to provide wastewater collection service to Village 1. New trunk sewers are shown on **Figure 6-3**.

Trunk A is proposed to be 9,300 LF and extend from the existing Regional Sewer Trunk, south in Oak Tree Lane. There is approximately 100 feet of elevation change from the upstream end of this sewer down to the Regional Sewer Trunk tie in location. Trunk B is proposed to be approximately 6,530 LF and extend northeast from the existing Regional Sewer, at the intersection of Ferrari Ranch Road and Hwy 193. The total elevation change along Trunk B is approximately 40 feet. A creek crossing may be required at the upstream end of the trunk depending on how far it is extended into Village 1. Trunk C is an extension of the existing Regional Sewer Trunk in Hwy 193, it is proposed to be approximately 1,280 LF and is needed to provide service to collector sewers in Stardust Lane. Infrastructure requirements for Village 1 are summarized in **Table 6-2**.

Table 6-2 Village 1 Proposed Infrastructure Summary

Pipeline ID	Pipe Size (in)	Slope (ft/ft)	Length (LF)	PWWF (MGD)
Village 1 Trunk – A1	12	0.003 – 0.0125	8,000	1.2
Village 1 Trunk – A2	15	0.0015	1,300	1.4
Village 1 Trunk – A3	12	0.002 – 0.01	3,400	0.2
Village 1 Trunk – B1	12	0.002	2,500	0.4
Village 1 Trunk – B2	15	0.0015	4,000	1.0
Village 1 Trunk – C, Regional Sewer	18 (1)	0.003 – 0.004	1,300	2.0

(1) Pipe size to be confirmed with City or County to accommodate actual flow from Bickford Ranch, etc.

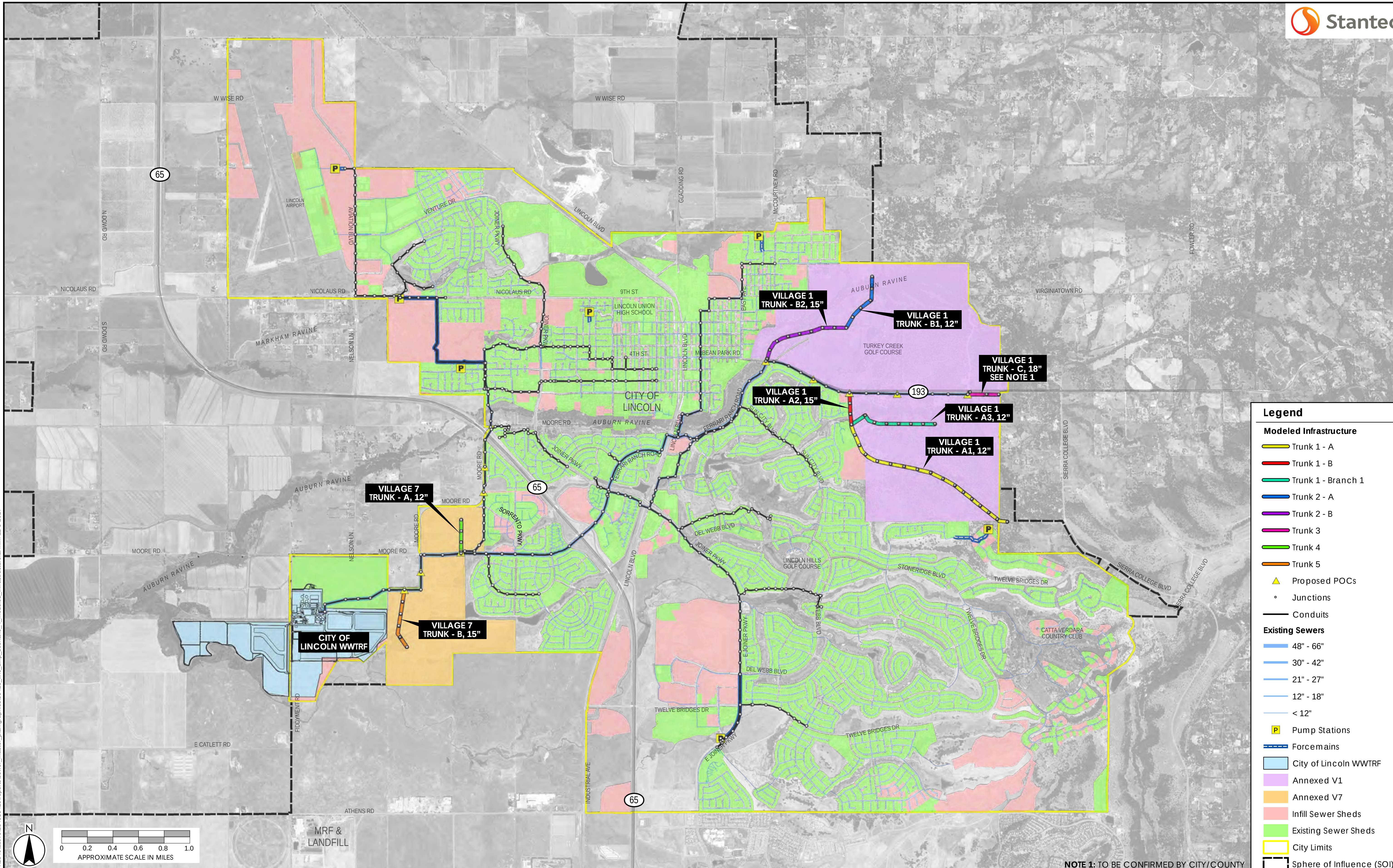

Figure 6-3

Scenario 4 – Modeled Infrastructure for Village 1 and Village 7

Infrastructure Required to Provide Service to Village 7

The location of Village 7 allows many of the collector sewers, proposed in the Village Specific Plan, to connect directly to existing trunk sewers. There are two proposed Point of Connections (POCs) along the Moore Rd Trunk sewer and three along the Regional Sewer trunk. Only two of the proposed sewers from the Specific Plan are 12-inch in diameter or larger and warrant inclusion in this Master Plan. Trunk A is proposed to be approximately 1,330 LF long and extend from the existing confluence of the Moore Road trunk and Regional Sewer trunk. The depth of the Regional Sewer provides sufficient elevation drop for Trunk B to service this area. Trunk 5 is proposed to be approximately 2,500 LF in length and extend south from the Regional Sewer trunk. Ground elevation falls on the south side of the Regional Sewer, but the depth of this POC is adequate to provide service to this area of Village 7. Infrastructure requirements to provide service to Village 7 are summarized in **Table 6-3**.

Table 6-3 Village 7 Proposed Infrastructure Summary

Pipeline ID	Pipe Size (in)	Slope (ft/ft)	Length (LF)	PWWF (MGD)
Village 7 Trunk – A	12	0.010 - 0.015	1,400	0.3
Village 7 Trunk – B	15	0.0018	2,500	1.4

6.3.3 Scenario 5 – Buildout of the SOI

The remaining portions of the SOI are proposed to be serviced by new trunk sewers that bypass the existing collection system and carry flow from Villages and SUDs to the WWTRF. The future trunk network is presented in **Figure 6-4**. Flow from the remaining portions of the SOI will not impact the existing collection system. A PWWF of 93.3 MGD is predicted to occur at the WWTRF. The LOS model results for Scenario 5 are included in **Appendix D**

A new 60-inch influent sewer will extend from the WWTRF to the intersection of Moore Road and Fiddyment Road to the WWTRF. A new 54-inch trunk in Nelson Lane will serve as the main trunk sewer providing service to the northern portion of the SOI and the NRPS sewer-shed through a CIP project further described in **Chapter 7.0**. The 42-inch Moore Road Trunk will provide service to Village 5/SUD-B, Village 6, and SUD-C. The new Nicolaus Road Trunk will provide service to Village 4 and SUD-A, and a small portion of Village 5/SUD-B. A total of 50 new trunk sewers are proposed with this Master Plan to serve the City's anticipated growth. These trunk sewers, their pipe sizes, slopes, total lengths, and service areas are summarized in **Table 6-4**.

The ground elevation of SOI ranges from approximately 208 feet above sea level in the northeast, to 77 feet above sea level in the southwest. High elevation in the northeast allows the slope of trunks serving Village 2 and Village 3 to exceed minimum slope requirements. While the remaining trunks serving the western portion of the SOI will require minimum slopes. Slopes of less than 0.0008 ft/ft will be required for the Moore Road Influent Trunk, Moore Road Trunk, Nelson

Lane Trunk, and Nicolaus Road Trunk. Minimum slopes that allow flow velocity to reach 2 ft/s under full pipe flow conditions were assumed for these trunks.

These trunk sewers have been designed to serve a large development area but will initially only have sewer connections serving a portion of the area. Low flow velocity under initial conditions could result in solids deposition along shallow sloped sewers, requiring regular cleaning to prevent solids accumulation and possible odor issues. To mitigate solids deposition, ovoid shaped pipes or pipes with dry weather flow channels shall be included in the design of these large shallow sloped sewers.

As an additional precaution, flushing systems using manhole chambers and gates have been identified at key locations in the proposed collection system. These systems allow for detention of sewage with a quick release to flush solids deposited on the sewer invert during low velocities. Flushing systems can be very effective in minimizing the accumulation of solids and may not be needed once development produces adequate base flow.

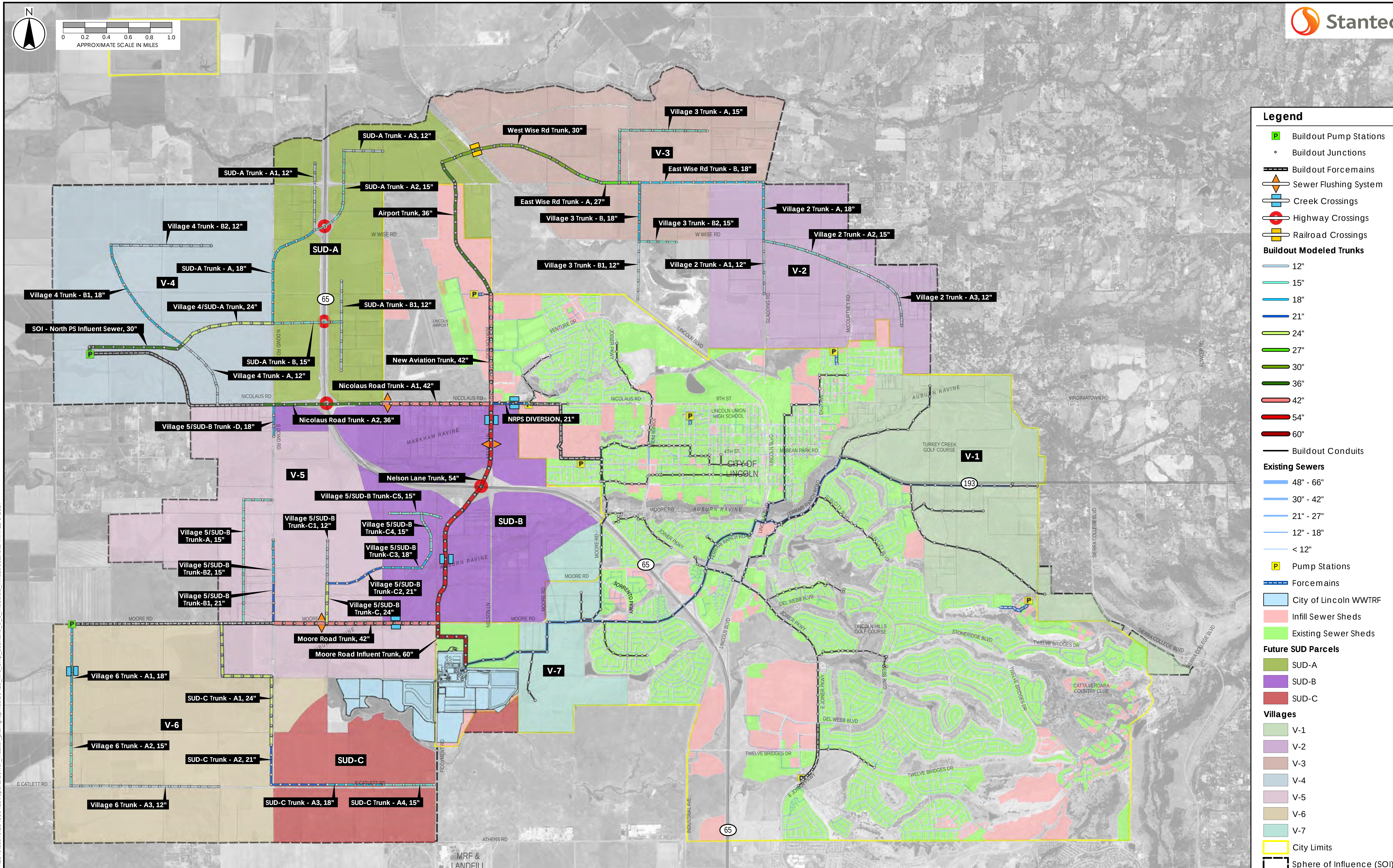

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Figure 6-4
Scenarios 5 and 6 – Modeled Infrastructure for Buildout of the SOI

Table 6-4 SOI Trunk Network Summary

Item	Description	Pipe Size (in) (1)	Slope (ft/ft) (3)	Length (LF)	PWWF (MGD)	Service Areas
1	Airport Trunk	24	0.0054	7,200	9.4	Village 2, Village 3, SUD-A
2	East Wise Road Trunk - A	27	0.0015	3,100	8.1	Village 2, Village 3
3	East Wise Road Trunk - B	18	0.0030	6,000	2.8	Village 2, Village 3
4	Moore Road Influent Trunk (2)	60	0.0003 n = 0.01(3)	3,400	39.8 33.0 (4)	SOI & NRPS Collection Shed (excludes Village 1 and Village 7)
5	Moore Road Trunk (2)	42	0.0004 n = 0.01(3)	8,000	11.8	Village 5/SUD-B, SUD-C, Village 6
6	Nelson Lane Trunk (2)	54	0.0005 n = 0.01(3)	11,500	27.9 21.0 (4)	Village 5/SUD-B, Village 2, Village 3, Village 4, SUD-A, NRPS Collection Shed
7	New Aviation Trunk	42	0.0008	5,400	12.6 9.4 (4)	Village 2, Village 3, SUD-A, NRPS Collection Shed
8	Nicolaus Road Trunk - A1 (2)	42	0.0004 n = 0.01(3)	5,400	10.9	Village 4, SUD-A, Village 5/SUD-B
9	Nicolaus Road Trunk - A2 (2)	36	0.0005 n = 0.01(3)	5,800	9.7	Village 4, SUD-A, Village 5/SUD-B
10	NRPS Diversion	21	0.0025	2,100	4.0	NRPS Collection Shed
11	SOI - North PS Forcemain	21/24	NA	9,900	8.5	Village 4, SUD-A
12	SOI - North PS Influent Sewer	36	0.0008	5,000	8.3	Village 4, SUD-A
13	SOI - South PS Forcemain	21/24	NA	9,800	7.0	Village 5/SUD-B, Village 6, SUD-C
14	SOI - South PS Influent Sewer	24	0.0010	8,300	5.2	Village 5/SUD-B, Village 6, SUD-C
15	SUD-A Trunk - A	18	0.0015	5,700	2.3	SUD-A
16	SUD-A Trunk - A1	12	0.0020	3,300	0.9	SUD-A
17	SUD-A Trunk - A2	15	0.0015	4,600	1.1	SUD-A
18	SUD-A Trunk - A3	12	0.0020	1,900	0.6	SUD-A
19	SUD-A Trunk - B	15	0.0020	3,400	1.6	SUD-A
20	SUD-A Trunk - B1	12	0.0023	4,400	0.6	SUD-A
21	SUD-C Trunk - A1	24	0.0010	8,300	4.6	SUD-C
22	SUD-C Trunk - A2	21	0.0015	4,700	3.9	SUD-C
23	SUD-C Trunk - A3	18	0.0020	2,800	2.1	SUD-C
24	SUD-C Trunk - A4	15	0.0020	2,500	1.1	SUD-C
25	Village 2 Trunk - A	18	0.0030	2,900	2.8	Village 2
26	Village 2 Trunk - A1	12	0.0045	2,600	0.6	Village 2

Item	Description	Pipe Size (in) (1)	Slope (ft/ft) (3)	Length (LF)	PWWF (MGD)	Service Areas
27	Village 2 Trunk - A2	15	0.0040	4,400	1.5	Village 2
28	Village 2 Trunk - A3	12	0.0050	4,300	0.7	Village 2
29	Village 3 Trunk - A	15	0.0020	6,800	1.5	Village 3
30	Village 3 Trunk - B	18	0.0020	2,900	2.8	Village 3
31	Village 3 Trunk - B1	12	0.0025	2,900	1.1	Village 3
32	Village 3 Trunk - B2	15	0.0020	1,900	1.3	Village 3
33	Village 4 Trunk - A	12	0.0020	3,600	1.0	Village 4
34	Village 4 Trunk - B1	18	0.0012	6,200	2.0	Village 4
35	Village 4 Trunk - B2	12	0.0020	4,900	0.8	Village 4
36	Village 4/SUD-A Trunk	24	0.0012	4,700	5.0	Village 4, SUD-A
37	Village 5/SUD-B Trunk - A	15	0.0020	7,400	0.6	Village 5/SUD-B
38	Village 5/SUD-B Trunk - B1	21	0.0009	2,000	1.3	Village 5/SUD-B
39	Village 5/SUD-B Trunk - B2	18	0.0012	2,100	0.8	Village 5/SUD-B
40	Village 5/SUD-B Trunk - C	24	0.0008	2,000	3.1	Village 5/SUD-B
41	Village 5/SUD-B Trunk - C1	12	0.0020	2,100	0.3	Village 5/SUD-B
42	Village 5/SUD-B Trunk - C2	21	0.0009	2,900	2.6	Village 5/SUD-B
43	Village 5/SUD-B Trunk - C3	18	0.0012	2,900	2.6	Village 5/SUD-B
44	Village 5/SUD-B Trunk - C4	15	0.0015	1,900	1.5	Village 5/SUD-B
45	Village 5/SUD-B Trunk - C5	15	0.0020	2,600	0.6	Village 5/SUD-B
46	Village 5/SUD-B Trunk - D	18	0.0012	1,600	0.4	Village 5/SUD-B
47	Village 6 Trunk - A1	18	0.0012	5,400	1.9	Village 6
48	Village 6 Trunk - A2	15	0.0015	2,600	1.1	Village 6
49	Village 6 Trunk - A3	12	0.0020	7,200	0.7	Village 6
50	West Wise Road Trunk	30	0.0015	7,100	8.1	Village 2, Village 3, SUD-A

(1) All sewers sized based on a Mannings n of 0.013.
(2) Ovoid or dry weather channels required, or as approved by the City Engineer, to address minimum flow velocities.
(3) Reduced Mannings n required for some sewer segments to provide higher velocities at minimum flows.
(4) PWWF without the NRPS diversion.

Two new pump stations will be required to provide service to the western portion of the SOI. The locations of the proposed pump stations are presented in **Figure 6-4**. The SOI – North Pump Station will collect wastewater from Village 4 and SUD-A. The SOI – South Pump Station will collect wastewater from Village 6, SUD-C, and a portion of Village 5/SUD-B. The SOI – North Pump Station will require a reliable pumping capacity of 8.5 MGD and the SOI – South Pump

Station will need a peak pumping capacity of 7.1 MGD. Both pump stations will require a 21 to 24-inch forcemain, or two 15 to 18-inch dual forcemains.

Five new creek crossings will be needed with the construction of the new trunk network.

- Two new 54-inch crossings will be needed for the Nelson Lane sewer to cross Auburn and Markham Ravines.
- The Moore Road Trunk will cross Auburn Ravine upstream of Fiddyment Road, requiring a 42-inch crossing.
- An 18-inch crossing will be required as the Village 6 trunk approaches the SOI – South Pump Station.
- The final crossing will be needed along the 18/21-inch trunk required to divert flow from the NRPS.

Topographical information and survey data collected for this Master Plan indicates that elevation clearance exists between creek inverts and planned sewer profiles to provide gravity sewer service through these locations. However, detailed surveys and engineering will be required to confirm the creek crossing details and construction techniques. Environmental permits will also be required and may impart additional construction requirements on these sewer creek crossings. Several railroad crossings and highway crossings have also been identified that will impose additional construction costs. Creek, highway, and railroad crossings are identified on **Figure 6-4.**

6.3.4 Scenario 6 – Buildout of the SOI, plus Regional Flows

Approximately 7.0 MGD of regional PWWF from the City of Auburn and Placer County's Bickford Ranch were added to the buildout system model evaluated in Scenario 5. A total PWWF of 100.3 MGD is predicted to occur at the WWTRF. The addition of regional wastewater flows has little impact on the existing system capacity. This can be observed while comparing the LOS model results for Scenario 5 and Scenario 6, presented in **Appendix D**. Regional flows will be conveyed through the existing Regional Sewer Trunk to the WWTRF. The infrastructure required to convey regional flows to the City of Lincoln collection system from these remote service areas was not evaluated as part of this Master Plan (regional pump stations and pipelines outside of the City's boundaries were not included in this planning effort).

6.4 PUMP STATION CAPACITY EVALUATION

The following section presents an evaluation of the reliable and maximum pumping capacities of the NRPS and the ELPPS compared to the peak inflows to each pump station. The reliable capacity was used to simulate pump station capacity within the hydraulic model. The reliable pumping capacity is defined as the capacity of the pump station, with the largest pump out of service. The NRPS and ELPPS both have sufficient reliable capacity to pump the peak ADWF simulated in Scenario 1.

6.4.1 Nicolaus Road Pump Station (NRPS)

The NRPS has sufficient reliable pumping capacity to convey PWWF from its collection shed under 10-year, 24-hour design storm conditions for the existing level of development. The reliable capacity simulated in the hydraulic model reflects changes made with the 2017 improvement project. A PWWF of 3.1 MGD is predicted to occur at the pump station in Scenario 2 (existing conditions). Surcharging in Aviation Boulevard, shown in HGL Profiles 4 and 5 on **Figure C-5** and **Figure C-6**, attenuates the PWWF that would be observed if this capacity constraint was eliminated (increased pipeline capacity).

A PWWF of 4.9 MGD is predicted to occur at the NRPS in Scenario 3, exceeding both the reliable and maximum pumping capacities. A higher peak inflow would be predicted if the SSO and flow attenuation in the Aviation Trunk was eliminated (increased pipeline capacity). If the pump station capacity was increased to meet the peak inflow predicted in Scenario 3, surcharging in the downstream portion of the Aviation Trunk, shown in HGL Profile 5, would be reduced and the total spill volume from Profile 4 would be reduced from 1.7 MG to 1.5 MG. Model simulation results for Profiles 4 and 5, with and without capacity improvements at the NRPS are presented on **Figure C-14** and **Figure C-15**.

Surcharging and SSOs are predicted to occur in Scenario 3 along HGL Profile 11, as presented in **Figure C-12**. SSOs could be eliminated in this portion of the sewer and surcharging would be greatly reduced with pump station capacity improvements, but pipeline capacity constraints still exist. HGL Profile 11 is presented in **Figure C-16,** showing a comparison of model results with and without capacity improvements at the NRPS. Downstream manholes NW318SS04 and NW318SS03, between this profile and the NRPS are predicted to exceed LOS criteria in both scenarios. Surcharging depicted in HGL Profile 12, shown on **Figure C-13,** would be eliminated with increased pump station capacity.

Surcharging in the 30-inch sewer downstream of the NRPS forcemain discharge, shown in Profile 8 and presented on **Figure C-9**, is exacerbated by an increase in pumping capacity at the NRPS**.** Backwater effects cause the surcharge depth to exceed LOS criteria in manholes NW388SS03, NW388SS02, and NW388SS01 along the 24-inch sewer. Manholes NW389SS23 and NW390SS03 also fail LOS criteria for surcharge depth with pump station capacity improvements. The extent of the increase in surcharge depth in Profile 8 is presented on **Figure C-17**. No other downstream capacity restrictions have been identified as a result of increasing the capacity of the NRPS. However, it is still recommended that this collection shed be diverted to the Nelson Lane trunk upon its construction.

6.4.2 East Lincoln Parkway Pump Station (ELPPS)

Under ADWF conditions for existing levels of development (Scenario 1), peak inflow to the ELPPS is within the reliable capacity of the existing pump station. Under PWWF conditions (Scenario 2), peak inflow to the ELPPS surpasses the reliable capacity of the pump station. The ELPPS cannot reliably convey the predicted PWWF and the influent sewers become surcharged and SSOs are predicted as a result. Surcharging in HGL Profile 6 is presented in **Figure C-7**. A PWWF of 3.2

MGD is predicted to occur at the pump station. Although the reliable capacity is insufficient, assuming both large pumps are in operation, the maximum pump station capacity can convey this PWWF, eliminating surcharging in influent sewers. Sewers downstream of the forcemain discharge location would not be impacted by an increase in pump station capacity to meet PWWFs.

The PWWF predicted to occur at the ELPPS increases to 4.7 MGD with the addition of flow from infill development in Scenario 3. This PWWF exceeds both the reliable and maximum pumping capacities of the existing pump station. Like Scenario 2, surcharging of the ELPPS influent sewers is a result backwater effects from insufficient pumping capacity and not related pipeline capacity constraints. An increase in pump station capacity would eliminate the predicted surcharging within this sewer-shed. The sewers between the forcemain discharge and the connection to the Regional Sewer in Ferrari Ranch Road feature steep slopes and high residual capacity. The HGL in this reach of the system would increase less than 4 inches if the pump station capacity was increased to match PWWF at buildout. No downstream LOS failures or capacity constraints would result from these capacity improvements. A comparison of model results for HGL Profile 6, with and without capacity improvements to the ELLPS are presented in **Figure C-18**.

A summary of the pump station capacity evaluation is presented in **Table 6-5**.

Table 6-5 Pump Station Capacity Evaluation Summary (1)

Pump Inventory	NRPS	ELPPS
Pump 1 - Large Pump	60 HP	60 HP
Pump 2 - Large Pump	60 HP	60 HP
Pump 3 - Small Pump	20 HP	45 HP
Reliable Capacity (2) (MGD)	3.4	2.7
Maximum Capacity (3) (MGD)	3.7	4.0
Scenario 1 Capacity Required (MGD)	1.2	1.0
Scenario 2 Capacity Required (MGD)	3.1	3.2
Scenario 3 Capacity Required (MGD)	4.9	4.7
Peak Measured Flow (MGD)	2.8	2.1

(1) Scenarios 4, 5, and 6 do not add additional flow to pump station collection sheds.

(2) Reliable capacity is the minimum capacity assuming failure of the largest pump in the pump station, reliable capacity assumes simultaneous operation of one large pump and the small pump.

(3) Maximum capacity assumes simultaneous operation of both large pumps.

7.0 IMPROVEMENTS TO ADDRESS EXISTING SYSTEM DEFICIENCIES

7.1 PURPOSE

The purpose of this chapter is to provide recommendations for capital improvements to the City's collection system that eliminate capacity constraints and provide sufficient capacity to accommodate PWWFs predicted to occur during a 10-year, 24-hour design rainfall event.

This chapter is divided into the following sections:

- Capacity Deficiencies
- Recommended Capital Improvement Projects
- I/I Reduction Strategies

7.2 CAPACITY DEFICIENCIES

This section provides a summary of the existing system deficiencies identified by the hydraulic model under existing (Scenario 2) and buildout (Scenario 3) PWWF conditions. Capacity constraints were identified in **Chapter 6.0** and are summarized in **Table 7-1**. Recommended Capital Improvement Projects (CIPs) consider worst case capacity deficiencies, which occur under buildout development (Scenario 3). Improvements will be proposed based on buildout flow requirements and LOS Criteria described in **Section 5.2**. CIPs presented in this section address all capacity constraints presented in the following table. Some of the capacity constraints identified below are alleviated by downstream improvements, which reduce backwater effects.

Table 7-1 Summary of Collection System Capacity Restrictions

Item	Trunk Sewer	Sewer-shed	HGL Profiles & Figures	Limiting Conditions	Capacity Limitation
1	Old Town North	Shed 5	Profiles 1 & 2 Figures: C-2 & C-3	Existing PWWF	• Inconsistent pipe size and slope • Insufficient capacity in the sewers upstream of Joiner Parkway
				Buildout Scenarios	• SSOs are predicted to occur in this section of the collection system; capacity limited sections remain the same with the addition of flow from infill development, total overflow volume increases
2	Old Town South	Shed 5	Profiles 3 & 9 Figures: C-4 & C-10	Existing PWWF	• Insufficient capacity in the 15-inch sewer upstream of Joiner Parkway.
				Buildout Scenarios	• Insufficient capacity in the 18-inch sewer in 1st Street and Chambers Drive.
3	Aviation	Shed 1	Profiles 4 & 5 Figures: C-5, C-6, C-14, & C-15	Existing PWWF	• Insufficient capacity in the 10-inch sewer upstream of the NRPS
				Buildout Scenarios	• Insufficient capacity in the 12-inch sewer in Aviation Boulevard • Insufficient reliable pumping capacity of the NRPS
4	North E. Street	Shed 3	Profile 7 Figure C-8	Existing PWWF	• Insufficient capacity in the 12-inch sewer upstream of the intersection of 12th Street and Hoitt Ave.
				Buildout Scenarios	• Insufficient capacity in the sewers upstream of the intersection of D Street and 9th Street
5	ELPPS Influent Sewers	Shed 10	Profile 6 Figures: C-7 & C-18	Existing PWWF	• Insufficient reliable pumping capacity of the ELPPS
				Buildout Scenarios	• No pipeline capacity restrictions, surcharging is the result of insufficient reliable pumping capacity at the ELPPS • SSOs are predicted to occur, surcharged pipe section remains the same
6	Lincoln Crossing	Shed 7A	Profile 10 Figure C-11	Buildout Scenarios	• Insufficient capacity in the 12-inch sewer with the addition of flow from Lincoln 270
7	Old Town North/ Moore Road	Shed 5/6	Profile 8 Figures: C-9 & C-17	Buildout Scenarios	• Insufficient capacity in the 30-inch sewer downstream of the confluence of the NRPS discharge, Old Town North Trunk and Old Town South Trunk
8	NRPS Collection Shed	Shed 1	Profiles 11 & 12 Figures: C-12, C-13, & C-16	Buildout Scenarios	• Insufficient reliable pumping capacity of the NRPS causes surcharging throughout the sewer-shed • Pipeline capacity restrictions exist in the Aviation Trunk (Item 3), and two sewers that convey flow to the NRPS from the collection shed upstream of Markham Ravine

7.3 RECOMMENDED CAPITAL IMPROVEMENT PROJECTS

Capacity constraints summarized in **Table 7-1** were identified under existing and future buildout PWWF conditions. All recommended pipe improvements are based upon the LOS criteria described in **Section 5.2**. The proposed CIPs provide sufficient flow capacity to convey flow from buildout development and PWWF conditions. CIPs are recommended in an order based on the current risk, extent of surcharge and overflow, and impacts on the upstream system. Actual development conditions and additional study of the existing system will need to be considered to predict the actual phasing and implementation of the proposed improvements. The recommended CIPs for the existing collection system are shown on **Figure 7-1**.

7.3.1 CIP 1: Old Town North Trunk

Based on the results of the existing hydraulic model simulation, improvements to both slope and pipe size should be made to address capacity deficiencies identified in the Old Town North Trunk. This CIP has been identified first because SSOs are predicted to occur under existing PWWF.

The existing collection system geodatabase shows a drop connection at NW388SS11, near the intersection of 5th street and Joiner Parkway. This elevation drop of 10-feet occurs as the trunk crosses Joiner Parkway. This drop connection could be reduced or eliminated to increase the available freeboard of the Old Town North Trunk. To accommodate a PWWF of 4.5 MGD the sewer should be upsized to a 21-inch between Countryside Drive and Joiner Parkway, and to 18-inches for the remaining portion of the improvements. A minimum slope of 0.004 ft/ft should be implemented along with the recommended pipe size improvements.

If added depth cannot be accommodated through the drop at manhole NW388SS11, and pipes are to be upsized in place or at the same elevation, 21-inch pipe will be required from NW388SS11 to NW423SS16 and 18-inch pipe will be required for the remaining improvements. A summary of the recommended improvements in CIP 1 is provided in **Table 7-2**.

Overflows are predicted to occur upstream of manhole NW457SS01 and capacity is limited in the upstream 12-inch pipe segment included in the model. Further study of this upstream collection shed is recommended to confirm the projected flow and capacity limitations that may exist upstream of its connection to the Old Town North Trunk.

Recommended improvements extend from manhole NW388SS11 to NW457SS01, as shown on **Figure 7-2**.

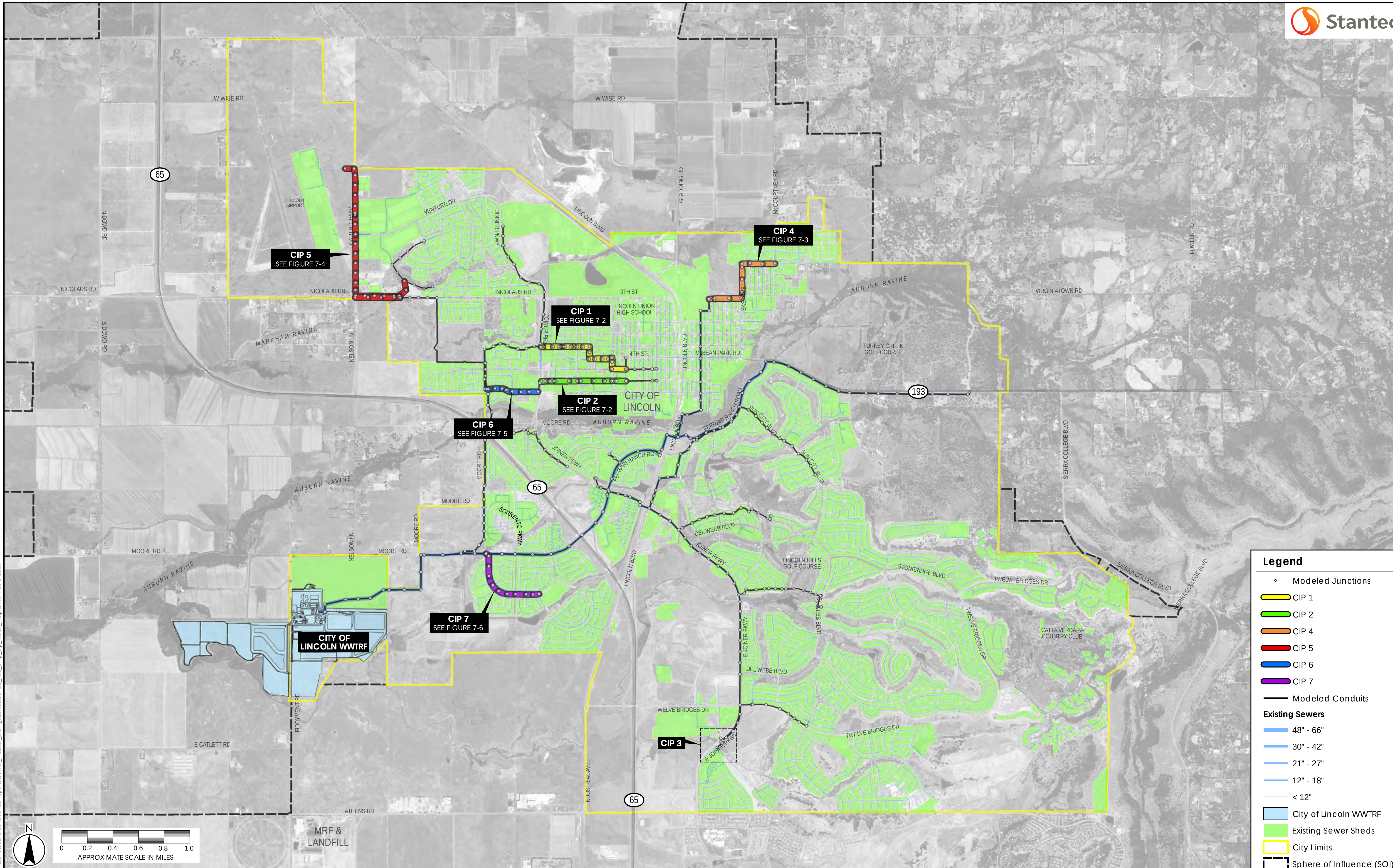
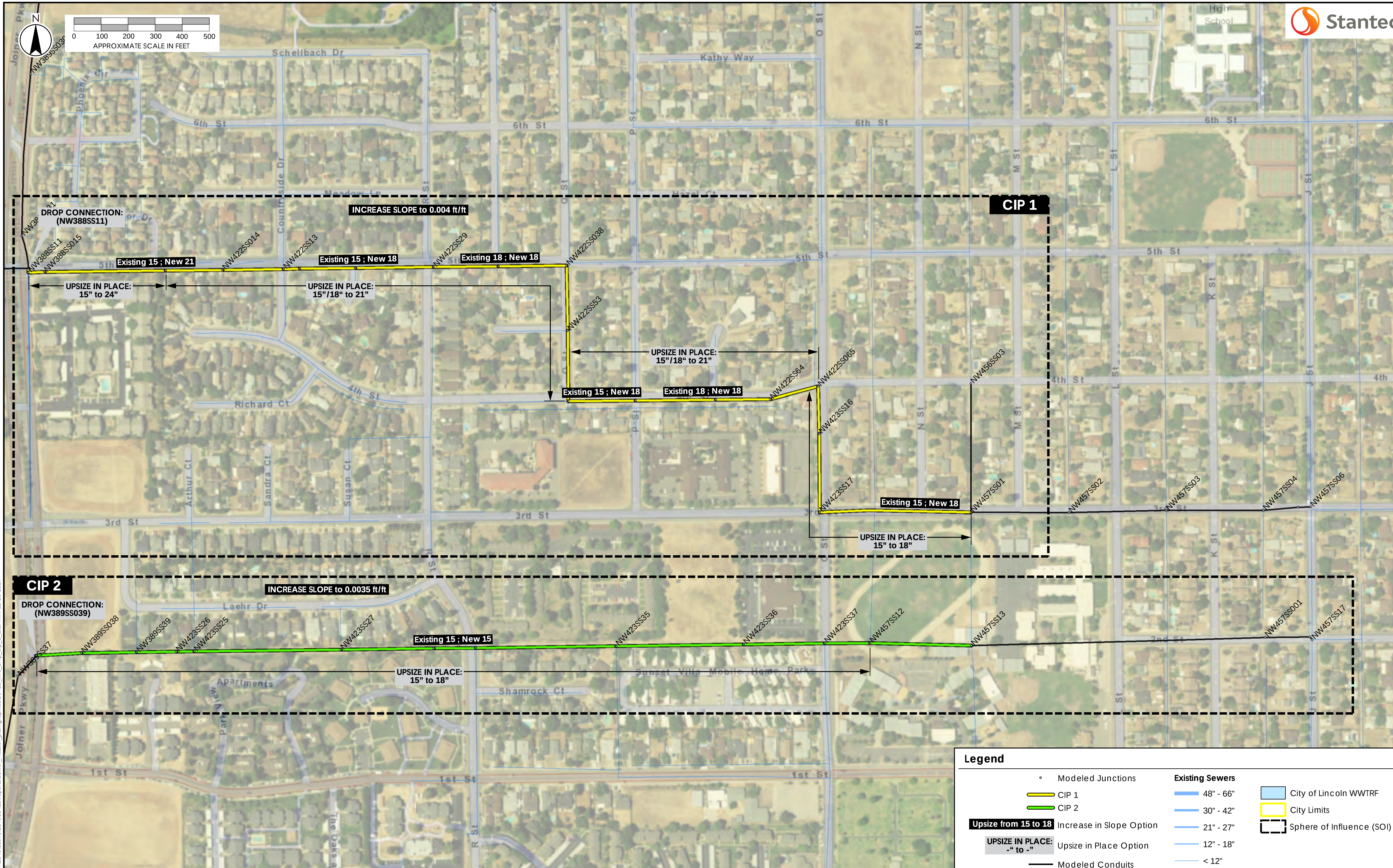


Figure 7-1
CIP Index Map



City of Lincoln
Wastewater Collection System Master Plan

Figure 7-2
CIP 1 and CIP 2

Table 7-2 CIP 1 Improvements Summary

Manhole ID	Existing Manhole Geometry			Existing Downstream Pipe Properties		Improved Manhole Geometry		Improved Downstream Pipe Properties		Upsize in Place
	Invert Elev. (ft)	Rim Elev. (ft)	Depth (ft)	Size (in)	Slope (ft/ft)	Invert Elev. (ft)	Depth (ft)	Size (in)	Slope (ft/ft)	Size (in)
NW457SS05 (1)	148.41	152.24	3.83	12	0.008	148.41	3.83	12	0.008	12
NW457SS04 (1)	147.39	153.94	6.55	12	0.007	147.39	6.55	12	0.007	12
NW457SS03 (1)	144.70	152.74	8.04	12	0.001	144.70	8.04	12	0.001	12
NW457SS02 (1)	144.26	151.83	7.57	12	0.001	144.26	7.57	12	0.001	12
NW457SS01	143.71	151.55	7.84	15	0.004	143.71	7.84	18	0.004	18
NW423SS17	141.58	150.21	8.63	15	0.005	141.58	8.63	18	0.005	18
NW423SS16	140.04	149.75	9.71	18	0.001	140.04	9.71	18	0.012	21
NW422SS065	139.84	149.04	9.19	18	0.001	137.92	11.12	18	0.004	21
NW422SS64	139.64	147.93	8.29	18	0.002	137.19	10.74	18	0.004	21
NW422SS62	139.16	147.15	7.99	18	0.002	136.38	10.77	18	0.004	21
NW422SS57	138.44	146.31	7.87	15	0.003	135.19	11.12	18	0.004	21
NW422SS55	137.81	145.54	7.73	18	0.003	134.09	11.45	18	0.004	21
NW422SS53	136.91	142.08	5.17	18	0.002	132.95	9.13	18	0.004	21
NW422SS038	136.48	146.66	10.18	18	0.004	131.88	14.77	18	0.004	21
NW422SS34	135.40	144.17	8.77	18	0.002	130.77	13.40	18	0.004	21
NW422SS29	134.99	141.58	6.59	15	0.003	129.71	11.86	18	0.004	21
NW422SS27	134.35	142.39	8.04	15	0.002	128.45	13.94	18	0.003	21
NW422SS26	133.90	142.63	8.73	15	0.003	127.74	14.90	18	0.003	21
NW422SS13	133.73	143.31	9.58	15	0.003	127.19	16.12	21	0.004	21
NW422SS014	133.09	143.99	10.90	15	0.003	126.18	17.81	21	0.004	21
NW422SS016	132.55	143.36	10.81	15	0.002	125.24	18.13	21	0.004	21
NW388SS015	131.51	138.99	7.48	15	0.000	123.34	15.65	21	0.004	21
NW388SS11	131.49	139.50	8.01	15	0.919	122.00	17.50	21	0.065	21

(1) Table entries are shown in profile figures, but do not require improvements.

7.3.2 CIP 2: Old Town South Trunk – Part A

It is recommended that improvements to both slope and pipe size be made to address capacity limitations identified in HGL Profile 3 along the Old Town South Trunk sewer. Surcharging of over 3-feet was predicted to occur in this reach of the collection system under existing PWWF conditions. Like the Old Town North Trunk, there is an existing elevation drop that occurs where the trunk meets Joiner Parkway, at manhole NW389SS039. This 7-foot drop should be reduced or eliminated to provide added depth and increased capacity to the Old Town South Trunk. The slope of the 15-inch trunk sewer should be increased from 0.0015 to a minimum of 0.0035 ft/ft to accommodate a PWWF of 2.2 MGD. If it is not possible to utilize the elevation drop, pipe sizes from manholes NW389SS039 to NW457SS13 should be increased to 18-inches. **Table 7-3** presents a summary of the recommended improvements included in CIP 2. Recommended improvements extend from manhole NW389SS039 to NW457SS12, as shown on **Figure 7-2**.

Table 7-3 CIP 2 Improvements Summary

Manhole ID	Existing Manhole Geometry			Existing Downstream Pipe Properties		Improved Manhole Geometry		Improved Downstream Pipe Properties		Upsize in Place
	Invert Elev. (ft)	Rim Elev. (ft)	Depth (ft)	Size (in)	Slope (ft/ft)	Invert Elev. (ft)	Depth (ft)	Size (in)	Slope (ft/ft)	Size (in)
NW457SS16 (1)	145.06	155.69	10.63	15	0.0014	145.06	10.63	15	0.0014	15
NW457SS001 (1)	144.83	154.44	9.60	15	0.0014	144.83	9.60	15	0.0014	15
NW457SS13 (1)	143.35	155.30	11.95	15	0.0016	143.35	11.95	15	0.0038	15
NW457SS12	142.75	154.94	12.19	15	0.0010	141.91	13.03	15	0.0035	18
NW423SS37	142.59	153.63	11.04	15	0.0015	141.33	12.30	15	0.0035	18
NW423SS36	142.13	153.55	11.42	15	0.0018	140.27	13.28	15	0.0035	18
NW423SS35	141.30	155.72	14.42	15	0.0015	138.62	17.10	15	0.0035	18
NW423SS31	140.52	149.78	9.26	15	0.0016	136.80	12.99	15	0.0035	18
NW423SS009	140.26	148.25	7.99	15	0.0016	136.25	12.00	15	0.0035	18
NW423SS27	139.72	148.51	8.79	15	0.0017	135.05	13.47	15	0.0035	18
NW423SS25	138.82	148.10	9.28	15	0.0014	133.15	14.95	15	0.0035	18
NW423SS26	138.73	147.65	8.92	15	0.0021	132.91	14.74	15	0.0035	18
NW389SS39	138.42	146.41	7.99	15	0.0014	132.40	14.01	15	0.0035	18
NW389SS038	138.14	144.88	6.73	15	0.0014	131.68	13.20	15	0.0035	18

(1) Table entries are shown in profile figures, but do not require improvements.

7.3.3 CIP 3: Pumping Capacity of the ELPPS

Surcharging and SSOs are predicted to occur in the influent sewers of the ELPPS under existing PWWF conditions (Scenario 2), which assumes only the reliable pumping capacity would be available. The reliable pumping capacity of the ELPPS is currently 2.7 MGD, corresponding to the operation of the 45 HP pump and one of the 60 HP pumps. The maximum pumping capacity of the pump station is 4.0 MGD, corresponding to the operation of both 60 HP pumps. The PWWF conveyed to the pump station in the existing system model is predicted to be 3.2 MGD, therefore exceeding the reliable pumping capacity with the addition of flow from infill development. There are no pipeline capacity limitations in the upstream influent sewers. PWWF to the pump station observed during the 2017 Flow Monitoring Study was 2.1 MGD.

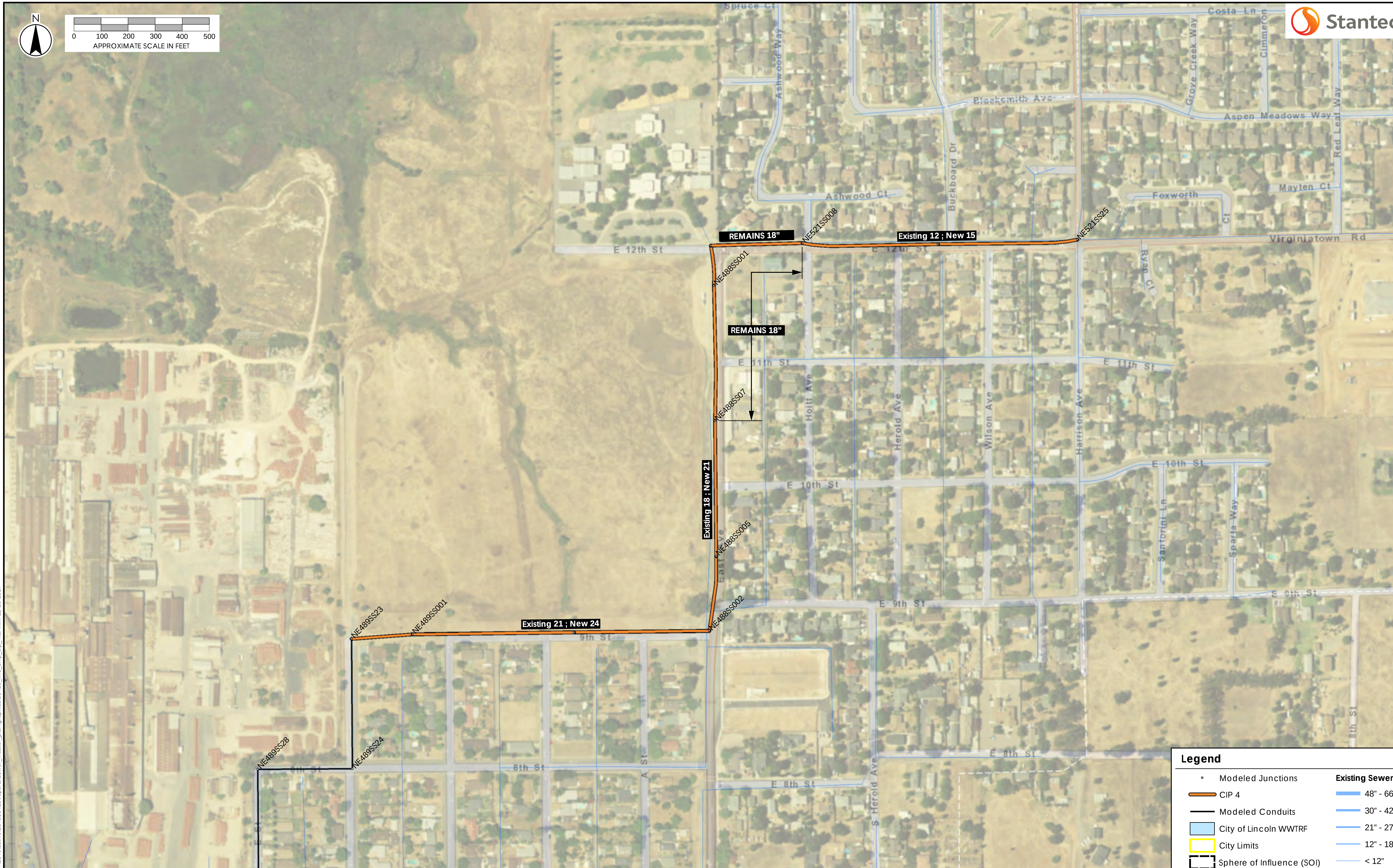
It is recommended that the reliable pumping capacity of the ELPPS be increased to meet the PWWF predicted in Scenario 3, under design storm conditions. There is available capacity in the dual 12-inch forcemains to convey the additional flow. Record drawings for the pump station suggest the ultimate replacement of the 45 HP pump with an additional 60 HP pump. This improvement would raise the reliable pump station capacity to 4.0 MGD, providing adequate capacity under existing wet weather conditions (Scenario 2). The ultimate reliable capacity of the pump station should be expanded to 4.7 MGD to convey PWWF with infill development (Scenario 3).

An emergency storage basin exists at this pump station. Although, manholes upstream of the pump station have lower manhole rim elevations than the pump station’s overflow elevation. It is recommended that these manhole rims be raised to a higher elevation than the pump station’s overflow elevation.

7.3.4 CIP 4: North E Street Sewer

The last two pipe segments along the E Street Trunk (of those included in the hydraulic model) were predicted to become surcharged under existing PWWF conditions. The inclusion of flow from infill development causes additional capacity limitations in the sewers immediately downstream, further exacerbating the predicted surcharge depth. The following improvements have been recommended to provide adequate capacity to convey flow under buildout conditions. It was assumed that pipes would be upsized in place and no slope improvements would be implemented.

The 21-inch sewer in 9th Street (between manholes NE489SS23 and NE488SS002) should be upsized to 24-inches, the first two segments of the 18-inch sewer in East Ave (between manholes NE488SS002 and NE488SS07) should be upsized to 21-inches, the remaining portion of the 18-inch sewer (between manhole NE488SS07 and NE521SS008) can remain 18-inches, and the 12-inch segments (between manhole NE521SS008 and NE521SS25) should be upsized to 15-inches. Recommended pipeline improvements included in CIP 4 are shown in **Figure 7-3** and summarized in **Table 7-4**.



City of Lincoln
Wastewater Collection System Master Plan

Figure 7-3
CIP 4

Table 7-4 CIP 4 Improvements Summary

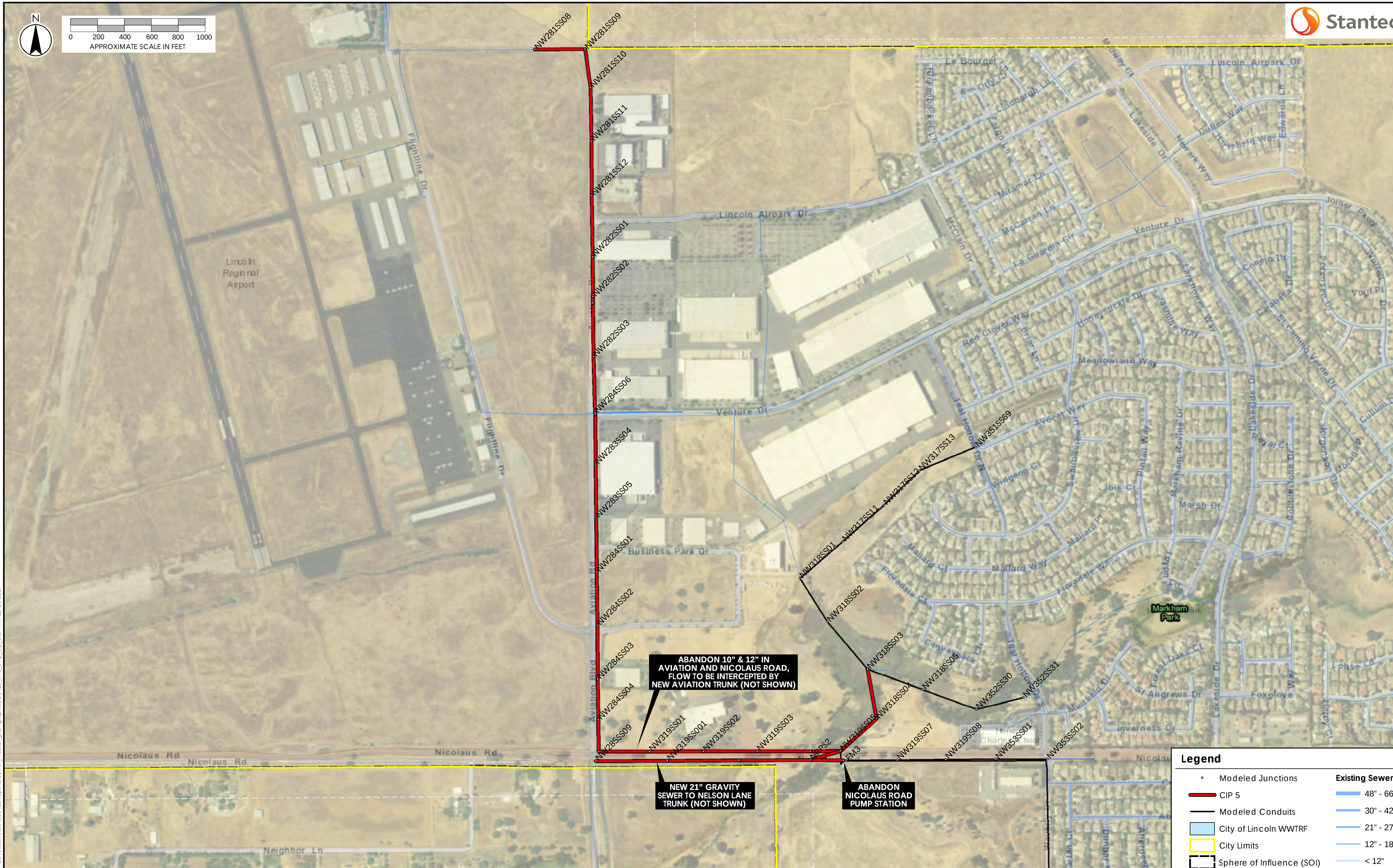
Manhole ID	Existing Manhole Geometry			Existing Downstream Pipe Properties		Improved Pipe
	Invert Elev. (ft)	Rim Elev. (ft)	Depth (ft)	Size (in)	Slope (ft/ft)	Size (in)
NE521SS25	168.36	182.97	14.61	12	0.0048	15
NE521SS52	165.89	176.00	10.11	12	0.0027	15
NE521SS008 (1)	164.02	171.30	7.28	18	0.0036	18
NE487SS03 (1)	162.80	170.40	7.60	18	0.0108	18
NE488SS001 (1)	161.19	171.50	10.31	18	0.0035	18
NE488SS07	159.35	175.37	16.02	18	0.0015	21
NE488SS005	158.50	174.19	15.69	18	0.0014	21
NE488SS002	157.75	174.72	16.97	21	0.0013	24
NE488SS006	157.00	171.93	14.93	21	0.0012	24
NE489SS001	156.26	173.77	17.50	21	0.0011	24
NE489SS23	155.46	173.53	18.07	27	0.0011	27

(1) Table entries are shown in profile figures, but do not require improvements.

7.3.5 CIP 5: NRPS Collection Shed Improvements

The proposed 54-inch sewer trunk in Nelson Lane is required to provide service to the northern SOI. It will extend north in Nelson Lane between Moore Road to Nicolaus Road. After it is construction, it is recommended that a CIP be implemented to divert flow from the NRPS collection shed, by gravity, to the new trunk. The implementation of this CIP will allow the NRPS to be decommissioned and alleviate capacity limitations imposed by the existing NRPS capacity and those identified in the downstream collection system. A new 21-inch sewer will be required to convey flow from the existing NRPS influent sewers to the new trunk in Nelson Lane.

The proposed 42-inch sewer in Aviation Boulevard is required to provide wastewater collection service to Village 2 and Village 3. This sewer will replace the existing 10-inch and 12-inch sewers identified as capacity limiting in HGL Profiles 4 and 5. Two new 18-inch sewers are needed to convey flow from manholes NW319SS05 and NW319SS06, to the new Nelson Lane Trunk sewer. In addition to the new sewers, it is recommended that two existing sewer segments are upsized with this project. The two 15-inch sewer segments upstream of NW319SS05, near Markham Ravine, should be upsized to 18-inches to eliminate surcharging under buildout PWWF conditions. These 15-inch sewers currently convey flow from HGL Profile 11 to the NRPS. The improvements needed to divert flow to the new Nelson Lane Trunk are shown in **Figure 7-4** and summarized in **Table 7-5**.



City of Lincoln
Wastewater Collection System Master Plan

Figure 7-4
CIP 5

Table 7-5 CIP 5 Improvements Summary

Manhole ID	Existing Manhole Geometry			Existing Downstream Pipe Properties		Improved Manhole Geometry		Improved Downstream Pipe Properties	
	Invert Elev. (ft)	Rim Elev. (ft)	Depth (ft)	Size (in)	Slope (ft/ft)	Invert Elev. (ft)	Depth (ft)	Size (in)	Slope (ft/ft)
NW318SS03	102.10	117.00	14.90	15	0.0033	102.10	14.90	18	0.0033
NW318SS04	100.86	117.13	16.27	15	0.0029	100.86	16.27	18	0.0030
NW319SS05	91.77	115.05	23.28	18	0.0113	99.77	15.28	18	0.0082
NRPS2	-	-	-	-	-	88.81	31.99	21	0.0025
NW319SS002	-	-	-	-	-	95.86	19.27	21	0.0025
NW319SS001	-	-	-	-	-	94.43	22.60	21	0.0025
NW285SS09	-	-	-	-	-	92.99	25.92	54	0.0005

The implementation of this CIP is dependent on the development and phasing of the future collection system. The pump station is currently at an acceptable level of service. An interim solution may need to be assessed to accommodate flow from infill development, prior to the construction of the new 54-inch Nelson Lane Trunk sewer.

7.3.6 CIP 6: Old Town South Trunk – Part B

Surcharging was predicted to occur in the Old Town South Trunk downstream of Joiner Parkway, as shown in HGL Profile 9. The existing 18-inch sewer flows west down 1st Street and turns north down Chambers Drive before meeting the confluence of the NRPS discharge and Old Town North Trunk sewer. The flow then back tracks south down Chambers Drive in the 30-inch sewer shown in HGL Profile 8. To reduce capacity constrictions in the 30-inch sewer and reduce the amount of upsizing needed along the Old Town South Trunk, it is proposed that the loop in Chambers Drive be eliminated by diverting flow from manhole NW389SS42 to manhole NW389SS23, near the intersection of Douglas Drive and 1st Street. Sewers upstream of manhole NW389SS42 should be upsized from 18 to 24-inches in diameter to eliminate surcharging.

Recommended improvements included in CIP 6 are shown on **Figure 7-5** and summarized in **Table 7-6**.

City of Lincoln
Wastewater Collection System Master Plan

Figure 7-5
CIP 6

Table 7-6 CIP 6 Improvements Summary

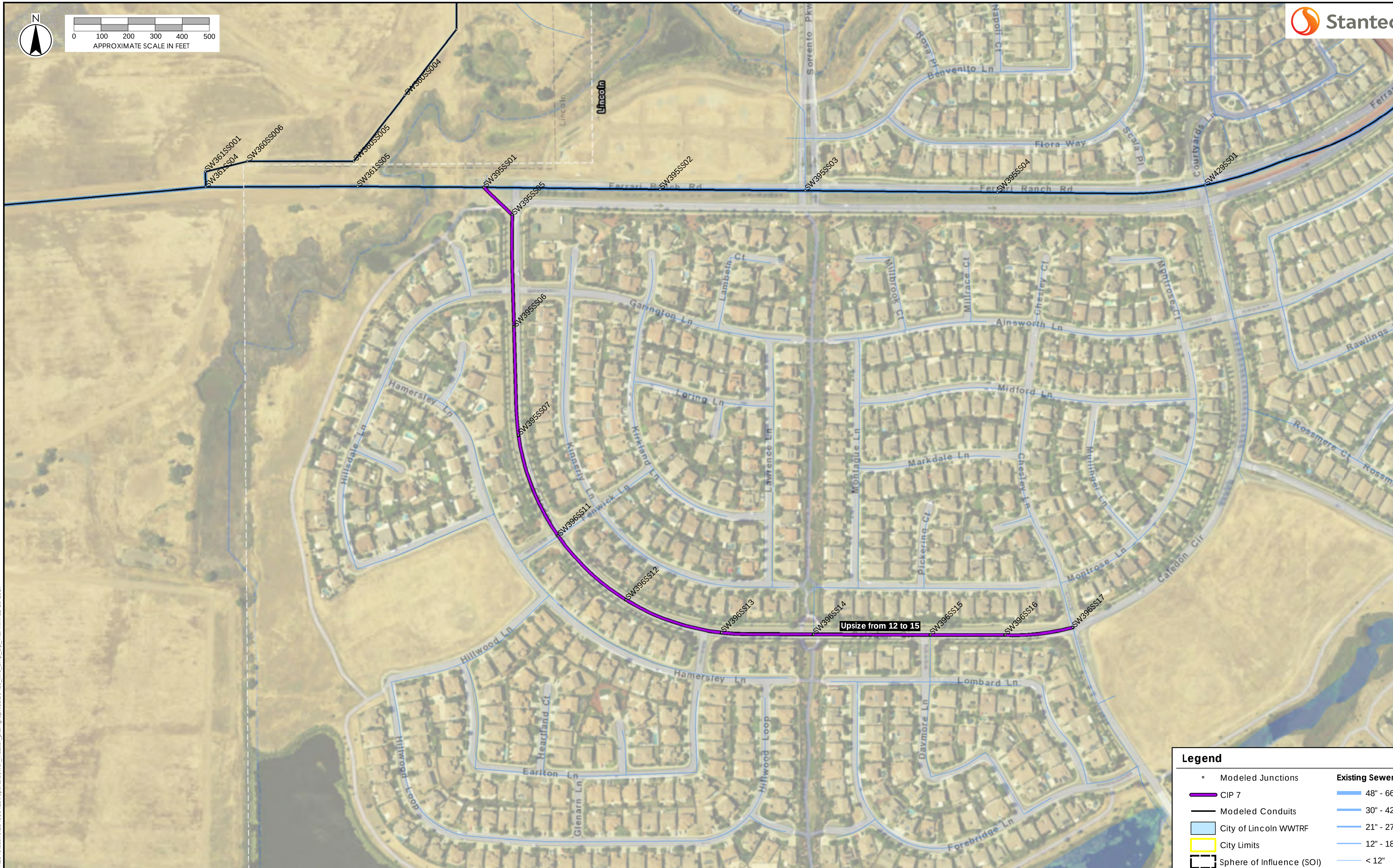
Manhole ID	Existing Manhole Geometry			Existing Downstream Pipe Properties		Improved Pipe
	Invert Elev. (ft)	Rim Elev. (ft)	Depth (ft)	Size (in)	Slope (ft/ft)	Size (in)
NW389SS37 (1)	128.01	144.21	16.20	18	0.0105	18
NW390SS21	124.36	146.00	21.64	18	0.0022	21
NW390SS20	123.57	139.68	16.11	18	0.0008	21
NW390SS19	123.29	137.41	14.12	18	0.0014	21
NW390SS18	122.78	137.60	14.82	18	0.0019	21
NW389SS33	122.30	138.20	15.90	18	0.0014	21
NW389SS31	122.04	138.70	16.66	18	0.0017	21
NW389SS25	121.55	135.00	13.45	18	0.0013	21
NW389SS42	121.20	135.10	13.90	18	0.0013	21
NW389SS24 (1)	121.12	135.00	13.88	18	0.0017	18
NW355SS49 (1)	120.95	134.89	13.94	18	0.0022	18
NW355SS29 (1)	120.07	132.74	12.67	18	0.0008	18
NW355SS19 (1)	119.75	130.80	11.05	18	0.0190	18

(1) Table entries are shown in profile figures, but do not require improvements.

7.3.7 CIP 7: Lincoln Crossing Trunk

The 12-inch Lincoln Crossings Trunk sewer has insufficient capacity to provide wastewater collection service to the Lincoln 270 Area under buildout PWWF conditions. The *Sewer Service Analysis for the Area Surrounding Athens Avenue,* (Stantec, 2017), recommends that this area be served by the proposed Athens Avenue Trunk sewer. The analysis considers routing flow from the Athens Area north along Fiddyment Road to the City of Lincoln's WWTRF, or south down Fiddyment Road to the City of Roseville's Pleasant Grove Wastewater Treatment Plant (PGWWTP).

The Lincoln Crossings Trunk sewer would need to be upsized from 12 inches to 15 inches, if wastewater collection service cannot be provided to the 270 Area by the Athens Avenue sewer. A pump station would be required to convey flow from the 270 Area to the point of connection in the existing collection system (upstream of those included in the hydraulic model). Pipelines between the point of connection and Ferrari Ranch Road would need to be upsized to serve the 270 Area. The improvements required to convey flow through the modeled Lincoln Crossing Trunk are shown on **Figure 7-6** and summarized in **Table 7-7**.



City of Lincoln
Wastewater Collection System Master Plan

Figure 7-6
CIP 7

Table 7-7 CIP 7 Improvements Summary

Manhole ID	Existing Manhole Geometry			Existing Downstream Pipe Properties		Improved Pipe
	Invert Elev. (ft)	Rim Elev. (ft)	Depth (ft)	Size (in)	Slope (ft/ft)	Size (in)
SW396SS17	108.26	129.30	21.04	12	0.0024	15
SW396SS16	107.65	129.40	21.75	12	0.0024	15
SW396SS15	106.99	128.00	21.01	12	0.0022	15
SW396SS14	106.02	128.10	22.08	12	0.0023	15
SW396SS13	105.24	126.90	21.66	12	0.0023	15
SW396SS12	104.39	126.00	21.61	12	0.0023	15
SW396SS11	103.60	125.80	22.20	12	0.0022	15
SW395SS07	102.69	127.30	24.61	12	0.0023	15
SW395SS06	101.79	127.60	25.81	12	0.0025	15
SW395SS05	100.76	128.10	27.34	12	0.0020	15
SW395SS01 (1)	96.97	126.70	29.73	54	0.0022	54

(1) Table entries are shown in profile figures, but do not require improvements.

7.4 I/I REDUCTION STRATEGIES

This section presents the results of an analysis of data collected during the 2017 flow monitoring study and per capita wastewater flows to determine if excessive I/I is occurring in the City's collection system. This section also presents strategies and recommendations to reduce I/I in the collection system. Excessive I/I is been defined by the California State Water Board (State Board) as an average daily flow generation rate that exceeds 120 gallons per capita per day (gpcd) during periods of sustained high groundwater, or having an average daily flow generation rate of more than 275 gpcd under wet weather conditions.

7.4.1 I/I Evaluation

Although V&A's 2017 Flow Monitoring Technical Report determined that excess groundwater infiltration occurs in Sheds 3, 5, 6, 9, and 10, the per capita wastewater generation rate does not exceed 120 gpcd during times of high groundwater infiltration. **Table 7-8** presents the average daily flow generation rate evaluation under periods of high groundwater infiltration.

Table 7-8 Average Daily Flow Generation Rate Evaluation, Peak ADWF

Item	Value
Peak Average Daily Flow During High Groundwater (MGD)	4.0
Approximate Population (excluding SMD1)	47,500
Approximate Peak ADWF Generation Rate (gpcd)	85

The peak average daily flow experienced at the WWTRF, under wet weather conditions, was 20 MGD, including SMD1. It was determined that approximately 8 MGD of this flow could be contributed to SMD1, leaving the City of Lincoln responsible for a peak average daily flow of approximately 12 MGD. The peak average daily flow during wet weather conditions was determined to be 253 gpcd, which is considered to be non-excessive. Results of this evaluation are shown in **Table 7-9**.

Table 7-9 Average Daily Flow Generation Rate Evaluation, PWWF

Item	Value
Peak Average Daily Flow During Wet Weather Conditions (MGD)	12.0
Approximate Population	47,500
Approximate PWWF Generation Rate (gpcd)	253

Although I/I within the collection system is not currently considered excessive by State Board standards, I/I in a collection system increases with system age. To maintain or reduce these levels of I/I, an on-going I/I reduction program, which includes improvement projects and control measures aimed at I/I reduction, is recommended.

Although 253 gpcd may be non-excessive for the City as whole, it is important to note that while SMD1 accounts for approximately 25 percent of the WWTRF's ADWF, it accounts for over 40 percent of the PWWF. It should also be noted that the majority of the City's existing collection system is less than 20-years old and "hot spots" or areas with high I/I exist within the collection system, principally in older areas, offering opportunity for localized I/I improvements within the City.

7.4.2 On-going I/I Reduction Program

The City may wish to conduct a study to determine whether it is more cost effective to locate the sources of I/I and systematically rehabilitate or replace faulty pipelines or continue to treat the additional flow at the WWTRF. The following on-going program is recommended:

1. **Improve Sewer System Database –** The database should include physical aspects of the system (pipes, manholes, laterals, etc.) as well as system assessment data (flow records, operation and maintenance logs, groundwater data, etc.)
2. **I/I Flow Monitoring Data –** Flow monitors should be strategically placed to further isolate areas with high I/I and evaluate the impacts of previously implemented I/I reduction projects. Data can also be used to keep the hydraulic model up to date and allow further refinement in unmonitored areas of the system.
3. **Sewer System Field Analysis –** This analysis involves an assessment of I/I sources through field activities using I/I isolation methods described below.
4. **System Remediation Plan –** This phase of the program considers rehabilitation or replacement project options to address sources of high I/I. Options should consider the structural and hydraulic conditions of the sewer system and future capacity requirements. Improvements identified for I/I reduction through rehabilitation or replacement, should be considered in addition to the CIP projects identified in this Master Plan. The CIP projects have been assessed based on capacity, not I/I reduction potential.
5. **System Remediation Plan implementation –** Implementation of the remediation plan entails designing and constructing the rehabilitation and replacement projects, monitoring the results and reviewing the plan as needed.

The following recommendations are provided for collecting I/I flow monitoring data:

- Determine flow monitoring locations based on the results of the previous flow monitoring studies, implement additional flow monitoring in areas with high I/I to further isolate sources.
- Groundwater infiltration in the 2017 flow monitoring data could be isolated by monitoring flow in each shed during dry weather months (July – September).
- Future wet weather flow calibration data should consider concurrently monitoring ground water levels or installing stream gauges throughout the City, which may correlate with infiltration, providing additional insights.

The following recommendations and investigation methods are recommended for the sewer system field analysis:

- Find and remove illicit flow from the collection system, such as improperly connected storm drains.
- Conduct structural assessments to establish the degree of physical deterioration of the sewer and to determine the proper rehabilitation or replacement technique, prioritizing aging infrastructure near ravines and drainage courses.
- Preform the following investigation methods:
  - Smoke Testing
  - CCTV Inspection/ Leak Location
  - Rainfall simulation/ Dye testing

The Reduction Program may also include non-technical measures to further reduce I/I, such as public education programs on sump pump or downspout connections, encouraging disconnection. After implementing I/I reduction projects determined by the Reduction program, the process should begin again by updating the sewer system database to include the improvements and data gathered throughout the process.

8.0 STRATEGIES FOR FUTURE SERVICING

8.1 PURPOSE

This chapter presents recommendations and approaches for addressing existing system deficiencies, providing interim service for infill development, and phasing of the future trunk sewer network to accommodate the projected sequencing of development of the City's SOI and connection of regional flow entities.

This chapter is divided into the following sections:

- Planning Scenarios
- Existing & Interim Collection System Needs
- Service Recommendations and Staging Strategies

8.2 PLANNING SCENARIOS

Results of system modeling suggest that portions of the existing wastewater collection system are not sufficiently sized to provide capacity for existing PWWFs or those projected to occur with infill development. By extension, these portions of existing system do not have sufficient capacity to serve development of the City's SOI. Providing service to these areas will require construction of several large trunk sewers, identified in **Section 6.3**. Financing, planning, and designing these improvements will take time. Depending on the order in which areas of the SOI develop, providing wastewater collection service to these areas will need to wait until major trunks are constructed, or interim solutions will need to be developed and implemented to accommodate SOI development. The predicted order in which development will occur was provided by the City and summarized in **Table 8-1**.

Table 8-1 Planning Scenarios

Planning Scenario	Cumulative Areas of Development
0	Existing (Including SMD1)
1	Infill Development
2	Recently Annexed (Villages 1 & 7)
3	Bickford Ranch
4	Village 5 + SUD-B
5	SUD-A, Villages 2, 3, and 4
6	SUD-C, Village 6 (Full Buildout)
7	Full Buildout w/ Auburn

8.3 EXISTING & INTERIM COLLECTION SYSTEM NEEDS

This section recommends initial steps in addressing existing capacity constraints within the City's collection system with limited new facilities or system upgrades. The most significant capacity restrictions exist within the northern portion of the collection system. Limited collection system improvements were identified within newer developments south of Ferrari Ranch Road.

8.3.1 Northern Collection System

There is insufficient capacity to convey PWWFs in the Old Town North Trunk, Old Town South Trunk, North E Street Trunk and the existing Aviation Trunk sewers. These trunks exist within flow monitoring sheds 1, 3, and 5, which all ranked in the top five of the ten sewer-sheds with the highest normalized I/I flow contribution. CIPs 1, 2, 4, 5, and 6 were identified in **Section 7.3** as planning level solutions for these deficiencies based on system modeling results. These are some of the oldest sewers in the existing collection system trunk network. Additional flow monitoring is recommended as an initial step in implementing these CIPs. Micro-monitoring within these collection sheds will provide data to further isolate areas with high I/I contributions and implement I/I reduction strategies recommended in **Section 7.4**. Flow monitoring within this area of the collection system should be used to confirm results predicted by hydraulic model simulations, from which CIPs have been developed.

Old Town and E Street Collection Sheds (Sheds 3 & 5)

Flow Monitoring Sheds 3 and 5 may have various manholes with pipe bifurcation, where flow may split and travel into two or more downstream pipes. The exact contribution of flow to a downstream flow monitor is dependent on the characteristics of these flow splits and their response to varying levels of flow. The current state of the hydraulic model assumes no pipe bifurcation occurs between Flow Monitoring Shed 3 and Shed 5. It is recommended that a more detailed delineation of the two sewer-sheds be performed with strategic flow monitoring to more accurately determine flow regimes between the E Street, Old Town North, and Old Town South Trunk sewers. Further, the existing geodatabase shows pipe slope and size inconsistences and many of the rim elevations were taken from LiDAR elevation data. Field verified invert and rim elevation surveys may be needed to update this information for use in the sewer-shed delineation. This information should be evaluated to refine results of the hydraulic model.

Diurnal flow patterns and I/I parameters simulated by the hydraulic model were generated for these trunks based on downstream flow monitoring data. Flow in the upstream portions of Sheds 3 and 5 may have different flow patterns or be further attenuated by pump stations that exist in the upstream, unmodeled collection system. Flow monitoring data for Shed 3 produced the highest proportion of I/I normalized by contributing area; addition of flow from infill developments may overestimate I/I predicted to occur within the model. Flow delineation and monitoring in the upstream portion of these trunks sewers is recommended with the detailed design of CIP 1, 2, 4, and 6.

NRPS Collection Shed (Shed 1)

CIP 5 proposes improvements which address capacity limitations within the NRPS collection shed (Shed 1). The timing of these improvements is dependent on the construction of the proposed 54-inch Nelson Lane Trunk and 42-inch Aviation Trunk. The following is recommended to the City to address Aviation Trunk capacity deficiencies under existing and interim conditions.

The 10-inch portion of the Aviation Trunk could be upsized to a diameter greater than 12-inches to reduce surcharging predicted to occur under existing conditions. Although surcharging is predicted to reach a depth of 3-feet above the pipe crown, the HGL is predicted to remain approximately 6-feet below the ground elevation. Until CIP 5 can be fully implemented, the City may choose to accept a lower level of service within the existing Aviation Trunk. Reducing I/I from this area could also provide additional capacity and further reduce the risk of SSOs[1].

The Aviation Trunk sewer was observed to have the highest normalized inflow contribution of the monitoring locations in the 2016 Flow Monitoring Study[2]. The timing of peak flow closely matched the time of peak rainfall intensity, suggesting that rainfall is entering the collection system directly through storm drain connections, open wet wells, inflow of surface runoff through leaky manhole lids, or other direct flow paths. It is recommended that the City inspect stormwater infrastructure at and around the Airport for improper connections to the collection system. Flow patterns from the Flight Line Drive pump station suggest that rainfall may be directly entering the pump station or upstream collection system.

8.3.2 Southern Collection System

The recommended improvements to the southern collection system have been identified as CIP 3 and CIP 7. CIP 3 proposes increasing the reliable capacity of the ELPPS and CIP 7 proposes upsizing the Lincoln Crossing Trunk to provide service capacity for the development of the Lincoln 270 Area. The ELPPS is fitted with an emergency storage basin and the maximum pumping capacity can convey existing PWWFs. Alternative servicing strategies have been identified for the Lincoln 270 Area (through the Athens Avenue sewer plan) and proposed improvements are dependent on the development of this area. Therefore, it is recommended that these improvements have a lower priority than those that exist in the northern portion of the collection system.

Of course, project timing can change priorities and recommendations. Project schedules should be reevaluated periodically to ensure that City infrastructure is in place ahead of project needs.

[1] The City has a current (2017) project to address I/I in this area, which may mitigate capacity concerns identified in this Master Plan.

[2] The high I/I in the Aviation Trunk may be due to the intake of stormwater from airport facilities. One solution would be to divert airport stormwater out of the sewer system. If stormwater is deliberately routed to the sewer due to water quality concerns, a stormwater detention and treatment basin, allowing for a conventional surface water release, would mitigate capacity impacts in the sewer.

8.4 SERVICE RECOMMENDATIONS AND STAGING STRATEGIES

The growth rate of development within the City's SOI will dictate the time horizon for buildout of the ultimate service area. However, it is possible that full buildout may not occur within the next 50 years. As a result, the City may elect to phase a portion of the new large trunk sewers. This section presents potential phasing strategies and guiding principles for planning the future collection system. Phasing strategies assume that development occurs in the order presented in **Table 8-1**. The City should work through possible options with stakeholders to come up with detailed phasing plans for the construction of new trunk sewers as new developments are proposed.

8.4.1 Phasing Strategies

Planning Scenarios 0, 1, 2, and 3: Infrastructure required to serve the development of areas within Planning Scenarios 0 through 3, as presented in **Table 8-1**, can be accommodated by direct connection to the existing system, assuming the implementation of the proposed CIPs. Ideally, the remaining Village and SUD areas are built outward from the WWTRF, such that they connect and are added to with subsequent development as growth radiates outward from the WWTRF. In this way, each development adds to the improvements completed by the previous development. Actual development conditions may not warrant this ideal approach for the implementation of new infrastructure. Therefore, the following phasing strategies have been recommended for the remaining Planning Scenarios.

Planning Scenario 4: Planning Scenario 4 assumes the development of Village 5/SUD-B, for which a Village Specific Plan has already been developed. This area is the closest to the WWTRF and its initial development will require the construction of the 60-inch Moore Rd Influent Trunk and 42-inch Moore Road Trunk sewers. To serve the northern portion of the area, 42-inch Nicolaus Road Trunk and the 54-inch Nelson Lane Trunk will need to be constructed. The developer may need to build these long sewers all the way from the WWTRF, at considerable expense, unless interim solutions are provided to serve the northern areas of the development. At this time, the City may elect to implement CIP 5, to divert flow from the NRPS to the new 54-inch Nelson Lane Trunk. Alternatively, the developer may propose to include a small pump station to serve the area under interim conditions.

The western portions of Village 5/SUD-B will ultimately be served by the two large SOI pump stations, but these large pump stations will be positioned at the western end of the SOI (western Village 4 and Village 6), to ensure their ability to serve the entirety of their respective collection sheds. If Village 5/SUD-B were to provide these large pump stations in their desired location, they would have to build a long sewer to the pump station, the pump station itself, and the forcemain to the WWTRF. This could create a cash-flow challenge for the developer, who may want to build a temporary pump station and forcemain until sufficient development is in place to support construction of the buildout infrastructure. The sewer drainages in these areas and the ultimate placement of temporary pump stations should be positioned to be easily decommissioned in the future and the areas served by gravity flow, when the ultimate sewer system is completed.

This Master Plan layout supports a Village 5/SUD-B collection system compatible with ultimately connecting to the future large pump stations but could allow the placement of small local pump stations for local service, only to be used in the interim. Then, in the future when additional development is in place and the large pump station is constructed, the interim pump stations can be decommissioned. This will allow for phased development of the infrastructure while minimizing the number of pump stations that the City must own and operate long term.

Planning Scenario 5: Planning Scenario 5 assumes the development of the remaining northern SOI area, including Village 2, Village 3, Village 4, and SUD-A. Considerable new infrastructure will be needed to provide wastewater collection service to these areas. The development of Village 2 and Village 3 will require the construction of the trunks within each respective Village area, the 30-inch West Wise Road Trunk, 36-inch Airport Trunk, the 42-inch Aviation Trunk, the 54-inch Nelson Lane Trunk, and the 60-inch Moore Road Influent Trunk.

The existing 30-inch E Street Trunk may be considered to route flow under interim conditions from the southern portion of Village 2 via temporary pump station. There is no residual capacity predicted to be available within the downstream end of the existing E Street Trunk Sewer under buildout conditions of its existing sewer-shed. The City may consider accepting a lower level of service of the 30-inch E Street Sewer under interim development conditions. However, the hydraulic model should be revisited to evaluate the impacts of specific interim conditions, before allowing additional flow to be routed through the E Street Trunk Sewer.

The development of Village 4 and SUD-A will require the construction of the large Northern SOI pump station and forcemain, the trunks within each respective development area, and the 36-inch/42-inch Nicolaus Road Trunk. As mentioned for the development of Village 5/SUD-B, interim service could be provided to areas within Village 4 and/or SUD-A using a small local pump station. This would require that the overall layout of the collection system is combatable with the layout of the ultimate collection system and allows these local pump stations to be decommissioned over time. Detailed survey and engineering designs may allow portions of SUD-A to connect by gravity to either the Nicolaus Road Trunk, Airport Trunk, or West Wise Road Trunk and should be considered at the time of development.

Planning Scenario 6: Planning Scenario 6 assumes the development of the remaining southern portion of the SOI, Village 6 and SUD-C. The development of Village 6 will require the construction of the large Southern SOI pump station and forcemain, and the trunks within Village 6. The same pump station and collection system ideology applies to this area of the SOI. A collection system that conforms to the overall layout of the future system and the installation of a temporary pump station may be considered to serve interim development. Development in SUD-C may consider alternative gravity alignments that connect to the WWTRF via Fiddyment Rood and should be considered with detailed survey and engineering designs.

Planning Scenario 7: Planning Scenario 7 assumes the connection of regional flow from the City of Auburn. Model simulations indicated that there is sufficient residual capacity within the existing Regional Sewer Trunk to accept regional flow from the City of Auburn. No phasing or development considerations are needed to accommodate this connection. Planning for infrastructure required to support this connection is outside the scope of this Master Plan.

8.4.2 Guiding Principals

Phasing strategies and overall planning of the future collection system should consider the following guiding principles.

Minimization of Pump Stations

Surface elevation data for the City's SOI was considered while developing layout of the future collection system. Large swaths of low lying land exist in the western portion of the SOI. One City Master Plan goal is the minimization of pump stations in the collection system. This is reflected in the inclusion of two large pump stations in the western SOI as part of the long term, buildout infrastructure, rather than multiple smaller pump stations around each Village and SUD. Although this can create a phasing challenge that may require interim solutions, it will reduce the overall buildout operating cost of the system. Ideally, interim solutions requiring temporary pump stations, can be decommissioned in the future and replaced with gravity service at a time when the temporary pump stations require new equipment or rehabilitation, minimizing stranded assets or unnecessary expenditures. This possibility will depend on the timing of future developments.

Maximization of Gravity Flow Sewers

Gravity flow alignments providing service to areas within SUD-A and SUD-C should be considered upon development. The Nicolaus Road and Moore Road Trunk sewers should be extended as far west as minimum slope and cover permits. Detailed survey and engineering designs will be needed to confirm how far west each trunk may be extended.

Minimization of Solids Deposition

Flushing systems may be needed to reduce solids accumulation in large trunk sewers upon initial development and use. Many of the large sewers trunks are designed to convey wastewater at a minimum velocity of 2 fps when flowing full. Upon initial construction, the trunks will not flow full, and low velocity and solids accumulation may occur. Flushing technologies that store and release flow should be considered to control odors due to solids accumulation and to minimize operation and maintenance requirements. Sewer pipe with low flow channels or ovoid shapes shall be considered with the initial design of the Nicolaus Road, Nelson Lane, Moore Road and Moore Road Influent trunk sewers, to be approved by the City Engineer. Three flushing locations are included in this Master Plan, strategically placed along these sewers. However, with City input, additional flushing stations should be considered with each project, as approved by the City Engineer.

Odor Control

Odor control should also be considered when developing designs for low slope sewers, flushing system, and pump stations. Preventing odor generation is good sewer design, but in some cases, where velocities are low, or sewage detention times are extended due to sewer length, odor control provisions should be considered, including proper venting, foul air treatment, and in the case of pump stations and forcemains, chemical addition can be considered.

Forcemains

Forcemains from the two proposed pump stations will require strategic planning during detailed design. To accommodate the probably low initial dry weather flow rates and the ultimate PWWFs at buildout, dual forcemains are recommended.

Aging Infrastructure

I/I within collection systems increases as pipelines age. The future collection system has been designed using a peaking factor of 2.3 to simulate peak wastewater flow. Peaking factors higher than 2.3 have been recorded within the existing system. Model simulations that applied existing I/I parameters to future infrastructure reduced flow attenuation in the upstream portions of the collection system, warranting larger pipe sizes. Upsizing pipelines that have been sized between 12 and 24-inches by one pipe size was needed to accommodate higher I/I associated with aging infrastructure. Therefore, the City should employ high sewer construction and inspection standards, operate a regular repair and replacement program and rehabilitate sewers, as needed, to minimize increases in I/I over time to maintain the LOS identified in this Master Plan.

9.0 CAPITAL COST ESTIMATES

9.1 PURPOSE

The purpose of this chapter is to present opinions of probable cost for the capital improvements recommended in this Master Plan. Planning level opinions of probable cost have been developed for the proposed CIPs and future SOI trunk network. These planning level estimates include construction costs, a 35 percent contingency for unforeseen conditions, and a 25 percent allowance engineering and environmental documentation. These costs have been estimated using the current ENR Construction Index (ENRCCI) of 11,013 (April 2018).

This chapter is divided into the following sections:

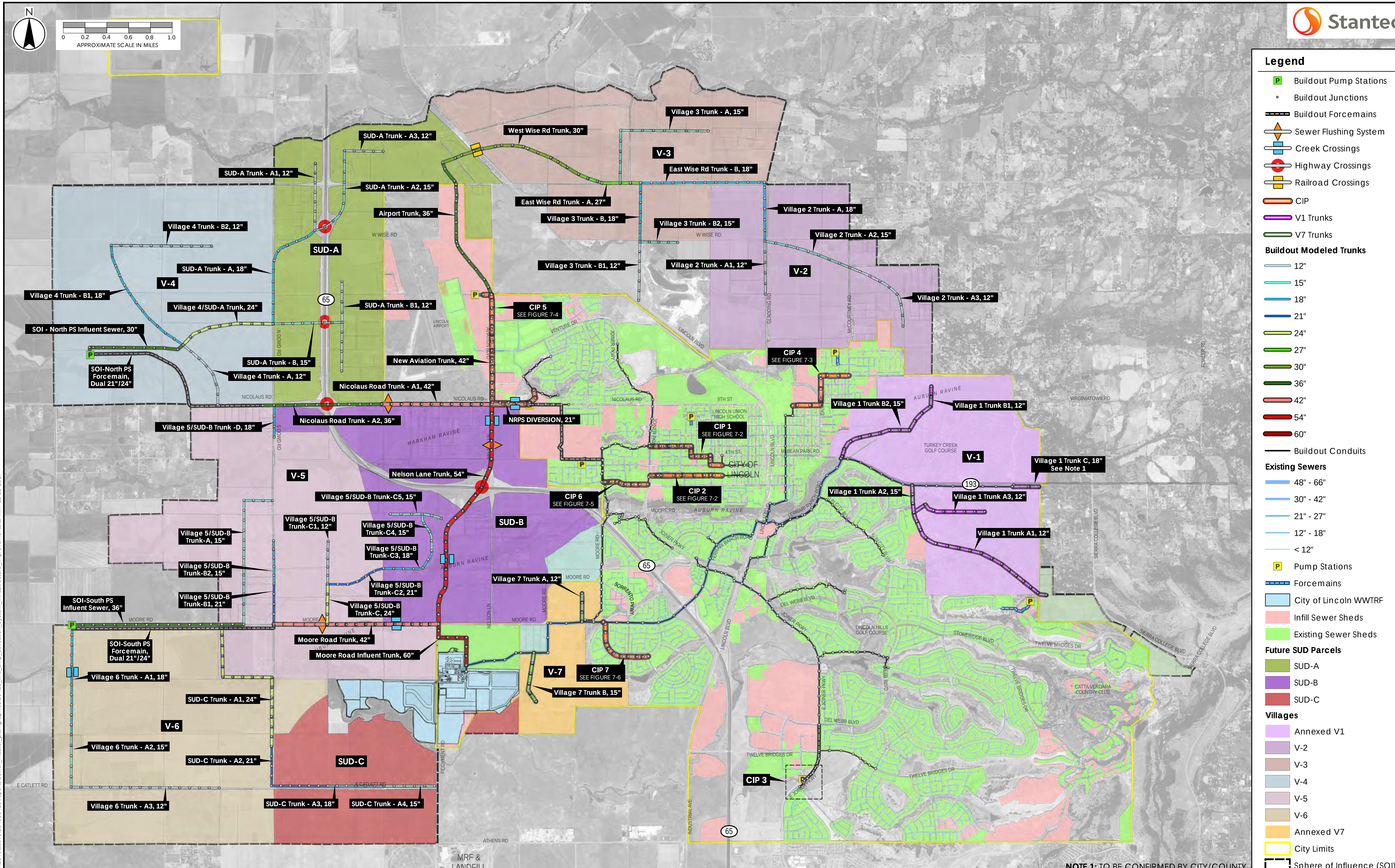
- Existing System CIP Costs
- Capital Costs to Serve the SOI

A figure showing the wastewater collection system improvements proposed in this Master Plan is included as **Figure 9-1**.

9.2 EXISTING SYSTEM CIP COSTS

Table 9-1 summarizes opinions of probable cost for CIPs proposed to address existing system deficiencies, presented in **Chapter 7.0**. These improvement projects do not include those that may be needed for repair and replacement (R&R) of aging infrastructure or wastewater projects currently identified in the City's existing 2017 – 2018 CIP budget, included in **Appendix F**.

A robust R&R program is a key element of any properly managed public infrastructure system and should remain in addition to these major capacity improvements referred to as CIPs 1-7. The City's R&R program includes an annual expenditure for the replacement of older, aging infrastructure. To replace all the sewers in the City's collection system would require a significant sum of money. The annual R&R allocation is intended to reduce the impact of repairing and replacing critical portions of the City's sewer collection system by stretching them out over time.



City of Lincoln
Wastewater Collection System Master Plan

Figure 9-1
Collection System Infrastructure Improvements

Table 9-1 Opinion of Probable CIP Costs

Item	Description	Construction Cost	Estimating Contingency (35%)	Engineering and Environmental (25%)	Total Project Cost
1	CIP 1 - Old Town North Sewer Improvements	$1,351,000	$473,000	$456,000	$2,280,000
2	CIP 2 - Old Town South Sewer Improvements	$992,000	$347,000	$335,000	$1,674,000
3	CIP 3 - ELPPS Reliable Pumping Capacity Improvements	$500,000	$175,000	$169,000	$844,000
4	CIP 4 - North E Street Sewer Improvements	$1,150,000	$403,000	$388,000	$1,941,000
5	CIP 5 - NRPS Gravity Sewer Diversion	$1,220,000	$427,000	$412,000	$2,059,000
6	CIP 6 - Old Town South Sewer Diversion and Improvements	$825,000	$289,000	$279,000	$1,393,000
7	CIP 7 - Lincoln Crossing Sewer Improvements	$1,198,000	$419,000	$404,000	$2,021,000
	Total				**$12,212,000**

(1) ENRCCI = 11,013, April 2018

In addition to these costs, interim improvements discussed in **Section 8.3** should be factored into the City's plans. Costs associated with the construction of the new 54-inch Nelson Lane Trunk, and 42-inch Aviation Trunk are not included in the CIP 5 – NRPS Gravity Sewer Diversion Project cost estimate. A breakdown of the construction costs of each CIP is provided in **Appendix B**.

9.3 CAPITAL COSTS TO SERVE THE SOI

9.3.1 SOI Pipelines

As discussed in preceding chapters, a new trunk sewer network is needed to provide service to the SOI. Over 50 new trunk sewer segments have been proposed, opinions of probable costs are presented in **Table 9-2**. A breakdown of the construction costs of each trunk sewer is provided in **Appendix B**.

Table 9-2 Village 1 and Village 7 Infrastructure Opinion of Probable Costs

Item	Description	Construction Cost	Estimating Contingency (35%)	Engineering and Environmental (25%)	Total Project Cost
1	Village 1 Trunk - A1 & A2	$2,660,000	$931,000	$898,000	$4,489,000
2	Village 1 Trunk - A3	$840,000	$294,000	$284,000	$1,418,000
3	Village 1 Trunk - B1 & B2	$2,080,000	$728,000	$702,000	$3,510,000
4	Village 1 Trunk - C	$480,000	$168,000	$162,000	$810,000
5	Village 7 Trunk - A	$470,000	$165,000	$159,000	$794,000
6	Village 7 Trunk - B	$900,000	$315,000	$304,000	$1,519,000
	Total				**$12,540,000**

(1) ENRCCI = 11,013, April 2018

Opinions of probable cost for the remaining trunks, lying outside of City Limits are presented in **Table 9-3**. A breakdown of the construction costs of each trunk sewer is provided in **Appendix B**.

Table 9-3 SOI Trunk Network Opinion of Probable Costs

Item	Description	Construction Cost	Estimating Contingency (35%)	Engineering and Environmental Documentation (25%)	Total Project Cost
1	Airport Trunk	$5,890,000	$2,062,000	$1,988,000	$9,940,000
2	East Wise Road Trunk - A	$1,900,000	$665,000	$641,000	$3,206,000
3	East Wise Road Trunk - B	$2,930,000	$1,026,000	$989,000	$4,945,000
4	Moore Road Influent Trunk	$4,450,000	$1,558,000	$1,502,000	$7,510,000
5	Moore Road Trunk	$5,970,000	$2,090,000	$2,015,000	$10,075,000
6	Nelson Lane Trunk	$15,050,000	$5,268,000	$5,080,000	$25,398,000
7	New Aviation Trunk	$5,920,000	$2,072,000	$1,998,000	$9,990,000
8	Nicolaus Road Trunk - A1	$4,860,000	$1,701,000	$1,640,000	$8,201,000
9	Nicolaus Road Trunk - A2	$3,300,000	$1,155,000	$1,114,000	$5,569,000
10	SOI - North PS Forcemain	$6,160,000	$2,156,000	$2,079,000	$10,395,000
11	SOI - North PS Influent Sewer	$3,680,000	$1,288,000	$1,242,000	$6,210,000
12	SOI - South PS Forcemain	$6,090,000	$2,132,000	$2,056,000	$10,278,000
13	SOI - South PS Influent Sewer	$5,090,000	$1,782,000	$1,718,000	$8,590,000
14	SUD-A Trunk - A	$2,310,000	$809,000	$780,000	$3,899,000

Item	Description	Construction Cost	Estimating Contingency (35%)	Engineering and Environmental Documentation (25%)	Total Project Cost
15	SUD-A Trunk - A1	$800,000	$280,000	$270,000	$1,350,000
16	SUD-A Trunk - A2	$1,390,000	$487,000	$469,000	$2,346,000
17	SUD-A Trunk - A3	$360,000	$126,000	$122,000	$608,000
18	SUD-A Trunk - B	$1,320,000	$462,000	$446,000	$2,228,000
19	SUD-A Trunk - B1	$1,320,000	$462,000	$446,000	$2,228,000
20	SUD-C Trunk - A1	$3,960,000	$1,386,000	$1,337,000	$6,683,000
21	SUD-C Trunk - A2	$2,190,000	$767,000	$739,000	$3,696,000
22	SUD-C Trunk - A3	$1,070,000	$375,000	$361,000	$1,806,000
23	SUD-C Trunk - A4	$760,000	$266,000	$257,000	$1,283,000
24	Village 2 Trunk - A	$1,420,000	$497,000	$479,000	$2,396,000
25	Village 2 Trunk - A1	$700,000	$245,000	$236,000	$1,181,000
26	Village 2 Trunk - A2	$1,490,000	$522,000	$503,000	$2,515,000
27	Village 2 Trunk - A3	$1,170,000	$410,000	$395,000	$1,975,000
28	Village 3 Trunk - A	$2,280,000	$798,000	$770,000	$3,848,000
29	Village 3 Trunk - B	$1,300,000	$455,000	$439,000	$2,194,000
30	Village 3 Trunk - B1	$720,000	$252,000	$243,000	$1,215,000
31	Village 3 Trunk - B2	$710,000	$249,000	$240,000	$1,199,000
32	Village 4 Trunk - A	$1,050,000	$368,000	$355,000	$1,773,000
33	Village 4 Trunk - B1	$2,220,000	$777,000	$749,000	$3,746,000
34	Village 4 Trunk - B2	$960,000	$336,000	$324,000	$1,620,000
35	Village 4/SUD-A Trunk	$2,890,000	$1,012,000	$976,000	$4,878,000
36	Village 5/SUD-B Trunk - A	$2,730,000	$956,000	$922,000	$4,608,000
37	Village 5/SUD-B Trunk - B1	$630,000	$221,000	$213,000	$1,064,000
38	Village 5/SUD-B Trunk - B2	$600,000	$210,000	$203,000	$1,013,000
39	Village 5/SUD-B Trunk - C	$790,000	$277,000	$267,000	$1,334,000
40	Village 5/SUD-B Trunk - C1	$560,000	$196,000	$189,000	$945,000
41	Village 5/SUD-B Trunk - C2	$1,150,000	$403,000	$388,000	$1,941,000
42	Village 5/SUD-B Trunk - C3	$1,030,000	$361,000	$348,000	$1,739,000
43	Village 5/SUD-B Trunk - C4	$580,000	$203,000	$196,000	$979,000
44	Village 5/SUD-B Trunk - C5	$710,000	$249,000	$240,000	$1,199,000

Item	Description	Construction Cost	Estimating Contingency (35%)	Engineering and Environmental Documentation (25%)	Total Project Cost
45	Village 5/SUD-B Trunk - D	$380,000	$133,000	$128,000	$641,000
46	Village 6 Trunk - A1	$1,780,000	$623,000	$601,000	$3,004,000
47	Village 6 Trunk - A2	$620,000	$217,000	$209,000	$1,046,000
48	Village 6 Trunk - A3	$1,540,000	$539,000	$520,000	$2,599,000
49	West Wise Road Trunk	$3,980,000	$1,393,000	$1,343,000	$6,716,000
50	Creek Crossings	$3,500,000	$1,225,000	$1,181,000	$5,906,000
51	Highway and RR Crossings	$3,500,000	$1,225,000	$1,181,000	$5,906,000
52	Large Trunk Flushing Systems	$1,500,000	$525,000	$506,000	$2,531,000
	Total				**$218,145,000**

(1) ENRCCI = 11,013, April 2018

9.3.2 SOI Pump Stations

Pump station cost estimates were scaled from similar projects in Placer County. The SOI – North Pump Station will need a peak pumping capacity of 8.5 MGD and the SOI – South Pump Station will need a peak pumping capacity of 7.1 MGD. For purposes of estimating cost, both pump stations were assumed to require a reliable capacity of 8.0 MGD. The cost estimate also assumes the following, a dual CIP wet well design, pigging station, backup generator, odor control, submersible pumps, and emergency storage basin. The probable cost of each pump station was approximated as $6,230,000.

9.3.3 SOI summary

A summary of the total costs to construct the future collection system is presented in **Table 9-4.**

Table 9-4 Total Cost for New Infrastructure (1)

Item	Description	Total Project Cost (2)
1	V1 & V7 Pipelines	$12,540,000
2	SOI Pipelines	$218,145,000 (3)
3	SOI Pump Stations	$12,460,000
	Total Cost:	**$243,145,000**

(1) ENRCCI = 11,013, April 2018

(2) Individual project costs include a 35% contingency and 25% allowance for engineering and environmental documentation.

(3) Additional cost information provided in Appendix B.