CDM Smith

Memorandum

To: *Paul Dawson, Marion Town Administrator*
Robert Zora, Marion Superintendent of Public Works

From: *Michael Guidice, P.E.*

Date: *June 30, 2017*

Subject: *Aucoot Cove Sewer Evaluation*
Preliminary Design Memorandum

Introduction and Background

The Town of Marion, in a joint partnership with the Town of Mattapoisett and the Buzzard's Bay Coalition (BBC), received a Southeast New England Program – Water Quality Management Grant (SNEP Grant) from the Commonwealth of Massachusetts Executive Office of Energy and Environmental Affairs Office of Coastal Zone Management based on a proposal submitted by the partnership entitled *Aucoot Cove Partnership to Reduce Nitrogen from Septic Systems* dated November 6, 2015. The intent of the project is to perform an evaluation of the feasibility of extending Marion's wastewater collection system to provide service to a total of approximately 158 existing homes in the Indian Cove (Marion) and Harbor Beach (Mattapoisett) neighborhoods and to provide secondary treatment at Marion's Wastewater Treatment Plant (WWTP); thereby eliminating on-site septic system discharges from these homes into Aucoot Cove.

Aucoot Cove is a shared Buzzards Bay waterway between Marion and Mattapoisett that fails to meet water quality standards due to excessive nitrogen pollution. Due to poor water quality, inner Aucoot Cove has been listed on the State's Integrated List of Impaired Waters (also known as "Dirty Waters List") as polluted by nutrients. Such waters require the establishment of a Total Maximum Daily Load ("TMDL") in order for those waters to meet their water quality standards. These TMDLs establish limits on the amount of nitrogen which can be discharged into these coves and also include a clean-up plan to restore water quality. While a TMDL evaluation has not yet been developed for inner Aucoot Cove, other estuaries in the region have received nitrogen threshold limits of 0.35 milligrams per liter (mg/L) total nitrogen. Total nitrogen levels in inner Aucoot Cove have shown concentrations above 0.35 mg/L since the early 2000s. Reducing sources of nitrogen will be essential for restoring water quality in Aucoot Cove.

Sources of nitrogen to marine waters typically include wastewater, stormwater, fertilizers and atmospheric deposition. The dominant source of nitrogen is wastewater, and the expansion of sewers to eliminate existing on-site septic systems and cesspools in the watershed is one of the most effective ways to reduce nitrogen. Existing on-site Title 5 septic systems typically discharge as much as 35 mg/L of nitrogen each. The homes in the Indian Cove and Harbor Beach neighborhoods collectively discharge an estimated total of approximately 5,300 pounds of nitrogen per year. If these homes were connected to Marion's WWTP, which discharges to Effluent Brook at the head of inner Aucoot Cove at approximately 4 mg/L, their nitrogen discharge could be reduced nearly 90 percent to a total of approximately 600 pounds per year.

The purpose of this memorandum is to summarize the results of CDM Smith's evaluation of the feasibility of sewering the Indian Cove and Harbor Beach neighborhoods in Marion and Mattapoisett, respectively. In accordance with the approved scope of work, the study included an evaluation of the capacity of the Marion WWTP to determine if the neighborhoods in the project area could be accommodated; evaluation of alternative collection systems to serve the areas; selection of a recommended collection system alternative; development of design plans for the recommended alternative (50 percent level for the Marion portion of the project area and 30 percent level for Mattapoisett); review of permitting requirements; cost estimates; and discussions of inter-municipal sewer service agreements and project financing.

Project Area

The project area includes the Indian Cove and Harbor Beach neighborhoods that lie along the western edge of Aucoot Cove in Marion and Mattapoisett, respectively. These areas were initially chosen for evaluation due to their proximity to Aucoot Cove, and the fact that they are not currently sewered and are relatively densely developed. The majority of other developed areas in Marion located in the Aucoot Cove watershed are already sewered (e.g., Olde Knoll neighborhood and Converse Road areas). However, there are additional adjacent areas in Marion that are in the Aucoot Cover watershed and are not currently sewered that could potentially be considered for sewering instead of, or in addition to, the Indian Cove and/or Harbor Beach neighborhoods. These areas include the stretch of Route 6 (Mill Street) north of Indian Cove Road up to Converse Road, Rocky Knook Lane, Sparrow Lane, Abels Way, Giffords Corner Road and Moorings Road. The project area as defined in the grant application is shown in **Figure 1**.

Indian Cove

The Indian Cove neighborhood in Marion is a private development off Route 6. The streets are owned and maintained by the Indian Cove Neighborhood Association and have not been accepted by the Town of Marion. There is a clubhouse that belongs to the homeowner's association located at the end of Indian Cove Road. The neighborhood consists of 29 existing single family homes (including the clubhouse) located on Indian Cove Road, Holly Pond Road, Alden Road and Sassamon Trail, which are all cul-de-sacs. There are also an additional three homes at the very end of Holly Pond Road to the north that are technically outside of the defined project area, but could potentially be connected to any proposed system that may be designed to sewer the Indian Cove neighborhood. The only vehicular access to the development is via Indian Cove Road from Route 6.

Harbor Beach

The Harbor Beach neighborhood in Mattapoisett consists of 129 existing single family homes located on Aucoot Road, Shore Drive, North Road, Center Drive, Harbor Road, Spruce Street, Cedar Street and Holly Street. Access to this neighborhood is via Aucoot Road, which is a long cul-de-sac, from Route 6. The houses in this neighborhood are immediately adjacent to the western shoreline of Aucoot Cove.

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Figure 1
Project Area

Other Adjacent Areas

As mentioned, there are several other unsewered streets in Marion that are within the Aucoot Cove watershed that are in the vicinity of the project area neighborhoods. These include Route 6, between Indian Cove Road and Converse Road; several cul-de-sacs off Route 6, including Rocky Knook Lane; Sparrow Lane; Abels Way; Giffords Corner Road; and Moorings Road, which is a dead end street off the end of Converse Road. This area of Marion includes approximately 150 existing unsewered properties; approximately 95 of which are in the Aucoot Cove watershed.

Current Plant Loading and Capacity Evaluation

CDM Smith completed an evaluation to determine if Marion's existing WWTP has the capacity to treat the increased flows and loads from an expansion of the collection system into the project area. The results of this evaluation show that the existing WWTP could handle an additional 251 typical residential connections, without modification, and still meet Marion's current National Pollution Discharge Elimination System (NPDES) permit limits. However, under the limits included in Marion's new Final NPDES permit issued on April 13, 2017, the WWTP could only accommodate an additional 78 typical residential connections. This decrease in capacity is based on the assumption that Marion may address its new phosphorous limit (0.2 mg/L) by adding chemicals to the treatment process. This chemical addition would result in an increase in sludge production and a corresponding decrease in treatment capacity in the existing sequencing batch reactors (SBRs) at the WWTP.

The new NPDES permit does give the Town the option of pursuing an extension of its existing outfall to the head of the salt marsh. This outfall extension to salt water would eliminate the phosphorous limit included in the new NPDES permit. By extending the outfall and thereby eliminating the need for chemical phosphorous removal, the WWTP would have capacity to add an additional 251 typical residential connections, as stated above. Therefore, the alternative that the Town decides to implement to address the phosphorus limit contained in their new NPDES permit will determine whether sewer extension into the project area (or a portion of the project area) is feasible based on the capacity of the WWTP. A separate technical memorandum that presents the results of the detailed evaluation performed on the WWTP capacity is included in **Attachment A**.

Topographic Survey

CDM Smith procured the services of Surveying and Mapping Consulting, Inc. (SMC) to perform surveying services for the project. These services included aerial photography, topographic mapping, collection and field verification of utility information and supplemental field surveys within the project area to develop the base plans used for the design of the collection system alternatives. The survey was prepared at a scale of 1-inch equals 40-feet, which is typical for this type of sewer design project and includes existing building information (house numbers and sill elevations), utility information and property lines, which were obtained from assessors maps. The datum used for the survey is the National Geodetic Vertical Datum of 1929 (NGVD 29).

Sewer Expansion Considerations

Several factors are typically considered during the preliminary planning of wastewater collection and conveyance system expansion, including, but not limited to: topography (maximize the use of gravity sewers); future flow estimates (used to size pipelines); estimated costs; subsurface conditions (manage installation cost by minimizing rock excavation); easements (maximize the use of right-of-way to limit easements); and permits (archaeological, wetlands, railroad, state highway, etc.). These factors help determine the appropriate collection system design for the area under evaluation.

Collection System Types

Collection systems are most frequently comprised of conventional gravity sewers, although topography and depth to bedrock generally dictate the sewer type and layout. Collection system alternatives considered during this evaluation included gravity sewers with pumping stations, low pressure sewers and a hybrid system (combination of gravity/pumped system and low pressure system).

Other potential types of collection systems include vacuum sewers and STEP (septic tank effluent pumping) systems. However, these systems were not considered feasible for this project. Vacuum systems have limitations in the amount of hydraulic head that they are able to overcome; typically 15 to 20 feet of hydraulic head. The project area includes a fairly significant elevation change from a low point of approximately elevation 5 on Shore Road in Mattapoisett to a high point of approximately elevation 27 on Indian Cove Road at Holly Pond Road in Marion. STEP systems operate similarly to traditional home septic systems, where the septic tank is retrofitted with an electric pump that pumps to a low pressure system and solids are left in the tank to degrade. However, due to the age of the existing on-site systems and the reported presence of cesspools in the project area, a STEP system would not be feasible for this project.

Existing Collection System

Marion's existing sewer collection system consists of approximately 19 miles of gravity sewers, eight pumping stations, 4.5 miles of force main and 8.5 miles of low pressure sewers and serves approximately 1,648 connections. In addition to the eight pumping stations, the Town is also responsible for the operation and maintenance of 430 grinder pumps installed as part of a sewer expansion project constructed in 2006. There is also one private pumping station and 59 private grinder pumps throughout the Town that are connected to the system but are not the responsibility of the Town.

There are several potential points for a sewer expansion in the project area to connect to Marion's existing collection system. The closest existing sewer is a privately-owned pressure sewer system in the Olde Knoll Road neighborhood. The system discharges flow from approximately 90 homes to a manhole on the Town's gravity system on Converse Road at its intersection with Zora Road. This system was reportedly sized to accommodate the connection of homes in the Indian Cove neighborhood. However, the existing system is not sized sufficiently to accept flow from the 158 homes in the project area. For the pressure sewer system alternative evaluated, connecting the entire project area to the Olde Knoll Road neighborhood would require either a complete replacement/upsizing of the existing system or alternatively, would require installation of a new pressure sewer dedicated to the new connections in the project area that would just traverse through the neighborhood and discharge to the manhole on Converse Road at Zora Road (or another manhole in that area), without connecting to the existing Olde Knoll Road system.

The manhole on Converse Road at Zora Road where the existing Olde Knoll Road system discharges would also be a potential connection point for a force main discharge from a new gravity/pumped system in the project area. To convey flow from the project area to this manhole would require a wetland crossing between Holly Road and Olde Meadow Road and new piping in Olde Knoll Road out to Converse Road. This would be a separate pipe emanating from the project area that would not be connected to the existing pressure sewer system in the Olde Knoll Road neighborhood.

Another potential connection point to the existing sewer system for a new expansion in the project area would be a manhole on a gravity sewer in Briggs Lane off of Route 6 (Mill Street). This connection point is further away from the project area than the manhole on Converse Road at Zora Road discussed above. However, use of this connection point would have the added benefit of potentially allowing future connection of some other unsewered properties in the Aucoot Cove watershed along Route 6 outside the project area, as discussed previously. There are approximately 84 existing properties with septic systems located on Route 6, Rocky Knook Lane, Sparrow Lane, Abels Way and Giffords Corner Road that are in the Aucoot Cove watershed and could potentially be connected to a new system from the project area (either gravity/pumped or low pressure system) if it is connected to the existing system at Briggs Lane via a force main or low pressure sewer in Route 6.

Topography

Generally, the topography of the Indian Cove neighborhood slopes down from a high point of approximately elevation 30 at Indian Cove Road's intersection with Route 6 to a low point of approximately elevation 7 near the end of the cul-de-sac. Similarly, Holly Pond Road slopes down from its intersection with Indian Cove Road at approximately elevation 26 down to approximately elevation 7 at the crossing of Aucoot Creek near the limit of the project area. Sassamon Trail slopes down from its intersection with Indian Cove Road at approximately elevation 22 to approximately elevation 17 near the end of the cul-de-sac.

In Mattapoisett, the Harbor Beach neighborhood generally slopes from North Road from approximate elevation 11 south down to Shore Drive, with a low point of approximately elevation 5. From its intersection with North Road at approximate elevation 7, Aucoot Road slopes up to a high point of approximately elevation 26. From this high point, Aucoot Road slopes down in a northerly direction to a low point at approximately elevation 12 near the end of the cul-de-sac.

Subsurface Conditions

Although subsurface investigations in the project area were not included in the scope of work for this evaluation, subsurface conditions are an important factor in the evaluation of alternative sewer collection systems. Based on visual and anecdotal evidence it is readily apparent that bedrock/large boulders are prevalent throughout the project area and will have a significant impact on construction costs for all of the collection system alternatives evaluated. Exposed bedrock outcrops are visible throughout the area as are large boulders adjacent to the roadways, apparently excavated during construction of the houses and roadways within the project area. Additionally, Marion DPW reports that rock was prevalent during installation of the water main in the Indian Cove neighborhood. A complete geotechnical evaluation of the project area, including a subsurface investigation program, would be the logical next step in the development of the design of any sewer collection system in the project area.

Sewer System Alternatives Analysis

As discussed, three alternative sewer collection systems were evaluated for potential implementation in the project area: a gravity/pumped system; a low pressure system; and a hybrid system (combination of gravity/pumped system and low pressure system). Additionally, several options of these alternatives were also evaluated during this study, including different routing scenarios/connection points and potential limits of sewering as discussed previously. To evaluate these alternatives, CDM Smith utilized the topographical survey information developed for this study, available GIS data for the project area and site visits/field reconnaissance. These field programs were used to identify areas of ledge and surface water, assess possible sewer pipeline routes, verify the existence of buildings, assess potential pumping station locations and to help evaluate future permitting that may be required for the selected alternative. The information gathered served as the foundation for all subsequent alternatives analysis, including development of cost estimates.

Various methods were used to choose the location of proposed sewers, force mains and low pressure sewers. Conceptual layouts were prepared based on GIS information and topographic mapping followed by field investigations to help confirm the layout of the sewer pipe, identify properties that cannot be served by gravity in a cost-effective manner and to identify potential alignments for cross country (off road) routes.

Generally, proposed sewer layouts were developed based on the concept perceived to be the best to provide service to the entire project area given the proximity of the connection points to the existing sewer system. Other approaches and layouts are feasible and could be considered as the project moves forward.

Alternative 1 - Gravity/Pumped Sewer Collection System

Gravity sewers are typically designed at a minimum depth of 10 feet below the sill elevation of existing homes to be served, where possible. Generally, this depth will allow first floors and most existing basement fixtures to be served by gravity. However, in particular cases where sill elevations are especially low, and bedrock is shallow, a decision may be made to serve first floor fixtures only. In extreme cases, individual buildings may be too low, in comparison to surrounding properties, to connect even first floors by gravity without driving the main line sewer excessively deep. These homes often require individual grinder pumps. Typically, gravity sewers are approximately 10 to 12 feet deep, however, this can vary considerably depending on the local topography. The minimum pipe size for a street sewer is 8-in diameter, with individual service pipes typically 6-in.

The conceptual gravity sewer system laid out to serve the project area (Alternative 1) is shown in **Figure 2** and consists of approximately 13,200 linear feet (lf) of 8-in sewer; 3 submersible pumping stations; and approximately 13,600 lf of 4-in force main. The system shown includes actual proposed manhole locations as well as potential locations for each of the three pumping stations

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Figure 2
Alternative 1 - Gravity and Pumped Sewer

that would be required due to the topography of the project area: one in the Harbor Beach neighborhood; one at the end of Indian Cove Road; and one on Holly Pond Road.

The conceptual location selected for the pumping station in the Harbor Beach neighborhood is on a privately-owned vacant lot on Harbor Road. There are actually three adjacent vacant parcels at this location (between house #13 and #19), so there would be more than sufficient space for the station. There are visible bedrock outcrops/boulders on these properties. It should be noted that this location is not the low point of the neighborhood, which lies on Shore Drive. A potential location on Shore Drive was initially investigated; however, due to the ground elevation (approximately elevation 5) and the 100-year flood elevation (elevation 21) the electrical controls for the pumping station would need to be elevated more than 16 feet above existing grade. The existing grade at the selected site on Harbor Road is slightly higher (approximately elevation 7), and the 100-year flood elevation is lower (elevation 14). Therefore, at the selected location, the electrical controls would need to be elevated approximately seven feet above existing grade. Although not ideal, this could be accomplished by creating an earthen mound to raise the electrical controls to the required elevation. The site could be enhanced with plantings, but would likely be met with opposition by the nearby residents. A permanent easement would be required for this pumping station location.

This pumping station would receive flow from all the gravity sewers in the Harbor Beach neighborhood as well as a portion of Aucoot Road south of the high point described previously. The pumping station would pump flow westerly via a 4-in force main in Harbor Road to Shore Drive, then northerly in Shore Drive and Holly Street. The force main would then run west in North Road onto Aucoot Road, where it would travel north to the high point in the road. At the high point, the force main would discharge to a manhole on an 8-in gravity sewer flowing northerly in Aucoot Road away from the high point. Near the cul-de-sac at the end of Aucoot Road, the gravity sewer would turn onto an existing gravel driveway that would lead to a second pumping station located in the vicinity of the cul-de-sac of Indian Cove Road.

The location shown in Figure 2 for this second pumping station is on the north side of Indian Cove Road near the homeowner's association clubhouse near the end of the cul-de-sac. This location was selected to minimize the height the electrical control panel would need to be elevated above existing grade to the extent possible. The selected location is at approximate elevation 11; the 100-year flood elevation is 17 in this area, which means the electrical controls would need to be elevated more than 6 feet above existing grade. Similar to the Harbor Road pumping station, this could be accomplished by creating a mound at the site. There are many very large boulders located on the side of the road in this area. In addition to receiving flow from the Harbor Beach neighborhood, this pumping station would also receive all the flow from the Indian Cove Road neighborhood via an 8-in gravity sewer in Indian Cove Road as well as an 8-in gravity sewer conveying flow from Alden Road and Sassamon Trail. A permanent easement would be required for this pumping station, which would serve as the main station for the project area, conveying all flow to the existing collection system. An easement would also be required for the gravity sewer between Sassamon Trail and the pumping station.

The force main from this pumping station would travel up Indian Cove Road and onto Route 6, then run north up to the discharge location at the existing manhole on Briggs Lane described previously. This force main alignment would allow for the potential future connection of additional properties along Route 6 and/or the adjacent streets which could connect to the force main via individual grinder pumps for each property.

The third pumping station in the conceptual gravity/pumped system alternative would be located on the north side of Holly Pond Road in the vicinity of house #7. The ground surface at this location is approximately elevation 10; the 100-yr flood elevation in this area is 17. Therefore, the electrical controls would need to be elevated more than 7 feet above existing grade, which could be accomplished by mounding. An easement would be required for this pumping station. This station would collect flow from seven houses on Holly Pond Road via 8-in sewers running in both directions and would pump flow via a 4-in diameter force main southerly up to Holly Pond Road's intersection with Indian Cove Road. At that point, the force main could connect to either a manhole on the 8-in gravity sewer in Indian Cove Road, or connect directly to the force main from the pumping station at the end of Indian Cove Road.

It is worth noting that along Aucoot Road in Mattapoisett there are a number of houses that are set back several hundred feet from the roadway (some up to 700 feet). In most cases, these homes would not be able to be served by gravity, since the setback distance would drive the main line sewer excessively deep based on the required slope of a gravity sewer service connection (minimum two percent slope required). Therefore, these homes would require individual grinder pumps to pump up to the gravity sewer in Aucoot Road for this gravity/pumped collection system alternative.

Alternative 2 - Low Pressure Sewer System

Low pressure sewers with individual house grinder pumps are often used as an alternative to conventional gravity sewers in areas of challenging topography (rolling terrain), to avoid construction of a municipal pumping station to serve a small number of homes, or to minimize construction cost due to shallow bedrock. Pressure sewers are most attractive and/or cost effective for sewering neighborhoods with less than 30 homes, in areas where a central pumping station cannot be easily sited and in areas with very shallow bedrock. Pressure sewers are also often used in areas served by gravity mains where several individual homes are too low to be served by the gravity system (or in the case of Aucoot Road, too far back from the road, as discussed above).

Pressure sewers are typically constructed of small diameter plastic pipe laid at relatively shallow depth (4 to 5 feet deep), whereby each property utilizes an individual grinder pump to discharge sewage from the property into the common pressure main. The grinder pump is typically housed in a buried plastic tank that provides storage capacity for the wastewater. Once the wastewater reaches a certain level, the grinder pump turns on and pumps into the common pressure main. Each individual grinder pump is typically isolated from the common pressure main with two check valves that prevent flow from being pumped back into individual grinder units as multiple units will be operating at any given time.

The conceptual low pressure sewer system laid out to serve the project area (Alternative 2) is shown in **Figure 3** and consists of approximately 23,000 lf of pressure sewer and 158 individual grinder

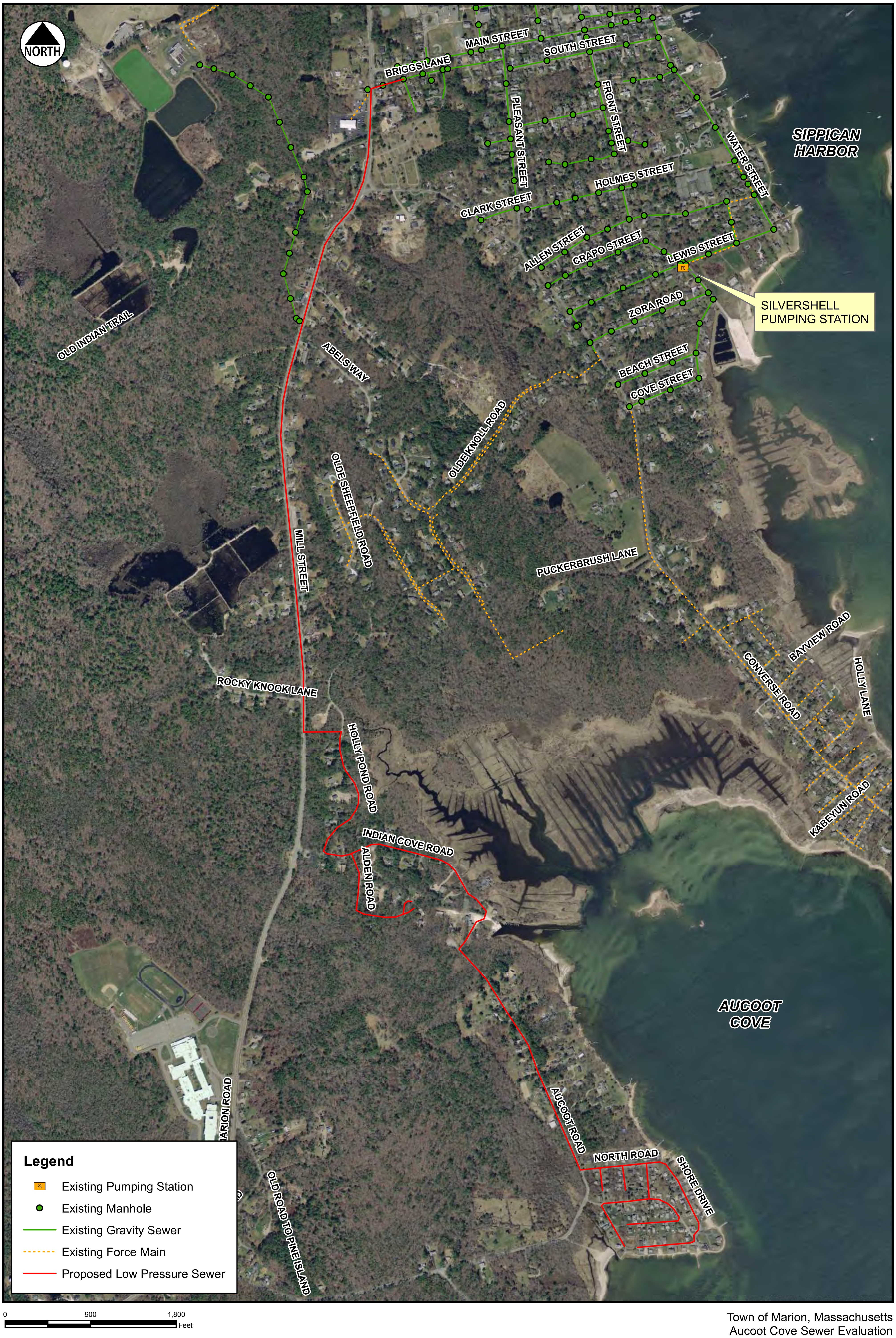

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Figure 3
Alternative 2 - Low Pressure Sewer

pumps. The pressure sewer pipe sizes include 5,300 lf of 2-in pipe; 2,620 lf of 3-in; 7,650 lf of 4-in; and 7,400 lf of 5-in pipe. As more homes are connected to the system, the size of the common pressure main increases to handle the additional flow. As shown in Figure 3, the pressure sewer layout begins in the Harbor Beach neighborhood, runs up Aucoot Road to Indian Cove Road, then travels down Holly Pond Road and cuts cross country out onto Route 6. The pressure sewer then runs north in Route 6 to its discharge at the manhole on the gravity sewer at Briggs Lane.

As discussed previously, this alignment would allow for the potential future connection of additional properties along Route 6 and/or the adjacent streets which could connect to the pressure sewer via individual grinder pumps for each property. For the additional properties to be connected, a portion of the 5-in pipe in Route 6 would need to be upsized to 6-in diameter in order to handle the additional flows.

Alternative 3 - Hybrid Sewer Collection System

Hybrid systems consist of a combination of two or more types of collection systems. Depending on the topography and layout of a proposed area to be sewered, it is sometimes most cost effective to combine different collection system types to serve different parts of a service area.

The conceptual hybrid system developed for the project area (Alternative 3) is shown in **Figure 4** and combines a gravity/pumped system with some pressure sewers. This alternative consists of approximately 12,100 lf of 8-in gravity sewer; 2 pumping stations; 13,300 lf of 4-in force main; and 1,400 lf of 2-in pressure sewer. This system consists mostly of the gravity/pumped collection system described for Alternative 1, with two changes. First, the gravity sewers and pumping station proposed for Holly Pond Road in Alternative 1 were replaced with a pressure sewer, since there were only seven homes being served by the pumping station. This pressure sewer would run southerly on Holly Pond Road to its intersection with Indian Cove Road and connect to the manhole on the gravity sewer at this location. The second change is the elimination of the gravity sewer on Alden Road and Sassamon Trail that included a cross country sewer running down to the pumping station proposed at the end of Indian Cove Road. This change eliminates a relatively long sewer that was only serving six houses and also eliminates the need to obtain several easements for the cross county portion of the sewer. The proposed pressure sewer would run up Sassamon Trail to Alden Road and discharge to a manhole on the gravity sewer at its intersection with Indian Cove Road.

The remainder of the Alternative 3 collection system would be as described for Alternative 1.

Planning Level Cost Estimates

Planning level opinion of probable construction cost estimates were developed for each of the three collection system alternatives described above to be used for comparative purposes to assist in selecting the recommended alternative to move forward to preliminary design. These planning level estimates were based on the conceptual lengths of pipe presented above; unit prices developed for the different pipe types/sizes (gravity sewer, force main and pressure sewer); additional costs to account for bedrock/boulder excavation for the various depths of pipe (assumed 10-ft depth for gravity sewers and 5-ft depth for force main and pressure sewer pipe); submersible pumping station costs (including bedrock/boulder excavation); and costs for grinder pump installation. The costs

0 900 1,800 Feet

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Figure 4
Alternative 3 - Gravity and Low Pressure Sewer (Hybrid)

include contractor overhead and profit and contingencies commensurate with the conceptual level of design development. The costs presented are in 2017 dollars and have not been escalated to the mid-point of construction since the implementation timeframe is not clear at this time.

The planning level opinion of probable construction costs for comparison of alternatives are as shown in **Table 1**.

Table 1 Planning Level Construction Cost

Alternative	Planning Level Construction Cost (Millions)
Gravity/Pumped Sewer Collection System	$8.1
Low Pressure Sewer System	$6.5
Hybrid Sewer Collection System	$7.7

Based on this analysis, Alternative 2 – Low Pressure Sewer System appears to be the least costly of the three conceptual collection systems evaluated.

Recommended Alternative

Based on the evaluation presented above, Alternative 2 – Low Pressure Sewer System is recommended to proceed to Preliminary Design. In addition to being the least costly alternative; the topography of the area, the subsurface conditions (shallow bedrock/boulders), the flood plain elevations (which would require elevated electrical controls for the pumping stations under Alternatives 1 and 3 as discussed above) and easement requirements for pumping station alternatives all favor a low pressure sewer system for implementation in the project area.

Preliminary Design

Design plans were developed for Alternative 2 - Low Pressure Sewer System based on the layout shown in Figure 3. A detailed hydraulic analysis of the layout and pipe sizing was performed by Environment One Corporation (E/One). In accordance with the scope of work, design plans were developed to the 50 percent level for the Marion portion of the recommended system (plan view and profiles) and to the 30 percent level for the Mattapoisett portion of the system (plan view only) and also include a cover sheet, general notes and legend sheet and a detail sheet. The plans show the layout of the high density polyethylene (HDPE) common pressure main, potential service connection locations for all properties in the project area, cleanouts/flushing structures and air/vacuum release valve locations. Design plans are included in **Attachment B**.

Easement Needs

Alternative 2 will require the acquisition of easements from four properties for the portions of the pipe alignment outside the roadways. Three easements would be required for the alignment between Aucoot Road and Indian Cove Road and one easement would be required for the alignment between Holly Pond Road and Route 6. Additionally, since Indian Cove Road and Holly Pond Road are private roads, the Town would either need to acquire utility easements within the roadways, or alternatively the Town could decide to accept the roadways as public prior to installation of the pressure sewers.

Refined Cost Estimate

A refined opinion of probable construction cost was developed for recommended Alternative 2 based on the design plans prepared, as discussed above and included in Attachment B. This estimate was developed using comparable publically bid projects and includes pressure sewer main line pipe, services, grinder pumps, provisions for rock/boulder excavation based on the excavation depth, paving, contractor overhead and profit, and contingencies commensurate with the level of design development. The costs are in 2017 dollars and have not been escalated to the mid-point of construction since the implementation timeframe is not clear at this time. The opinion of probable construction cost for Alternative 2 is $6.2M.

Funding Sources

The Clean Water State Revolving Fund (CWSRF) program is a low-interest loan program that can be used to fund construction projects that address water quality needs. The program is headed by the Environmental Protection agency (EPA) and jointly administered by the Massachusetts Department of Environmental Protection (MADEP) and the Massachusetts Water Pollution Abatement Trust. This program is self-funding, since previous loan payments – paid back into the fund – are redistributed to new projects during the next program cycle. Applications for the loans are solicited on an annual basis. In recent years, Massachusetts has annually distributed $300 million to $350 million in loans to 50 to 70 municipalities.

Projects are submitted to, and evaluated by, MADEP on an annual basis. Approved projects are added to a fundable list (Intended Use Plan), making that project eligible to submit a loan application in the next year. Eligible costs are submitted to MADEP, who must approve them before a binding loan commitment can be approved. Upon applying for and receiving a Project Approval Certificate, the borrower must commence the project within six months.

Municipal bonds can also be issued by the municipality to fund capital projects. The original investment and interest payments are paid back to investors on a regular basis over a fixed term of years. General obligation bonds are the most common type of bonds used by municipalities for capital projects. These bonds are not backed up by assets, so the investor must rely on the good faith of the municipality that they will receive a return. The rate of interest is based on risk assumed by the investor, which is rated as a credit rating by Standard and Poor's.

Permitting Review

For the purposes of performing a desktop permitting review for Alternative 2, the following assumptions were made:

- The majority of the preferred alternative route is located within existing paved or gravel roadways and outside of jurisdictional resource areas. This excludes an approximately 400 lf section of cross country sewer pipe proposed from Holly Pond Road to Mill Street next to #37 Holly Pond Road.

- The project will likely pursue Massachusetts Department of Environmental Protection (MassDEP) State Revolving Fund (SRF) loans, and therefore require additional permitting efforts triggered by the use of State funds.

Potential Environmental Permitting Constraints

For preliminary planning purposes, Massachusetts Geographic Information System (MassGIS) OLIVER (Mass OLIVER) was used to analyze recommended Alternative 2 for sewering the project area. The following resources were examined for potential permitting constraints:

- Wetlands;
- Floodplains;
- Estimated Habitat of Rare Wildlife and Priority Habitat of Rare Species;
- Historical and Archaeological Resources; and
- Coastal Zone.

Wetlands

The majority of the work proposed in the preferred alternative is located within existing paved and gravel roadways. A small portion, approximately 400 lf, is proposed cross-country from Holly Pond Road to Mill Street near #37 Holly Pond Road. According to Mass OLIVER, there do not appear to be any wetlands within or adjacent to this cross-country area.

However, although most project work will occur within existing roadways, there appear to be jurisdictional resource areas directly adjacent to portions of the project route. These jurisdictional resources include a perennial stream (Aucoot Creek) with an associated 200-ft Riverfront Area, Bordering Vegetated Wetlands (BVW), Salt Marsh and Coastal Beaches. The Alternative 2 may propose work located potentially within BVW and/or Salt Marsh, Inland Bank, 200-ft Riverfront area and within the 100-ft Buffer Zone of BVW and Salt Marsh.

Floodplain

The preferred alternative from Holly Pond Road, southward, is almost entirely within Zone AE as shown on the FEMA Flood Hazard Map. The work is proposed within Land Subject to Coastal Storm Flowage.

Estimated Habitat of Rare Wildlife and Priority Habitat of Rare Species

The preferred alternative from Holly Pond Road, southward, is almost entirely within Estimated Habitat of Rare Wildlife and Priority Habitat of Rare Species. Priority Habitat is defined as "*the geographic extent of Habitat for State-listed Species as delineated by the Division of Fisheries and Wildlife pursuant to 321 CMR 10.12. Priority Habitats are delineated based on records of State-listed Species observed within the 25 years prior to delineation and contained in the Division's NHESP database* (321 CMR 10:02)". Estimated Habitat of Rare Wildlife is a subset of Priority Habitats that are based on the geographical extent of habitat of state-listed rare wetlands wildlife and is codified under the Wetlands Protection Act. The proposed work within designated habitat is entirely within existing, paved roadways. According to 321 CMR 10.14 (6), construction of new sewer lines within paved roadway is an exempt activity and does not require Natural Heritage and Endangered Species Program (NHESP) review.

Historical and Archaeological Resources

The majority of work proposed in Alternative 2 is within existing paved and gravel roadways. It is not anticipated that the proposed project would impact any adjacent historical or archaeological resources. However, the project will be reviewed for historical and archaeological impacts during the future permitting phase to meet SRF requirements.

Coastal Zone

The entirety of the project area is within Coastal Zone Management (CZM) jurisdiction. Therefore, CZM review will be required as a part of this project.

Potential Federal Permits/Approvals

United States Army Corps of Engineers (USACE)

The USACE regulates Waters of the U.S. and their associated wetlands through Section 404 of the Clean Water Act. The potential work within BVW and/or Salt Marsh for the installation of new sewer lines within cross-country areas could potentially require approval by USACE. Cumulative impacts, including up to ½ acre (21,000 sf) of impacts to Tidal Waters of the U.S (excluding wetlands); 1,000 sf of impacts to Tidal Waters of the US within Special Aquatic Sites (SAS) (e.g., wetlands); and 100 sf of impacts to SAS including vegetated shallows qualify as a Pre-Construction Notification (PCN) General Permit (GP) 9 Activity (Utility Line Activities) [Sections 10 and 404] pursuant to the Massachusetts Department of Army New England GP, (effective date: March 9, 2015). A USACE PCN takes approximately one month to prepare and a maximum of 90 days to approve.

United States Fish and Wildlife Service

Section 10 of the Endangered Species Act (ESA) is designed to regulate a wide range of activities affecting plants and animals designated as Endangered or Threatened, and the habitats upon which they depend. With some exceptions, the ESA prohibits activities affecting these protected species and their habitats unless authorized by a permit from the U.S. Fish and Wildlife Service (USFWS) or the National Marine Fisheries Service (NMFS). Permitted activities are designed to be consistent with the conservation of the protected species.

Section 7 of the ESA requires Federal agencies to consult with the USFWS to ensure that actions they fund, authorize, permit, or otherwise carry out will not jeopardize the continued existence of any listed species or adversely modify designated critical habitats. The proposed project area is known habitat for several federally protected threatened or endangered species including the roseate tern (*Sterna dougallii*), the red knot (*Calidris canutus*), and the northern long-eared bat (*Myotis septentrionalis*). Coordination with USFWS will be required to confirm that the project will have “no effect” or is “not likely to effect” a federally listed threatened or endangered species. The coordination process can take approximately 30-60 days.

United States Environmental Protection Agency

The U.S. Environmental Protection Agency (EPA) regulates point source discharges of pollutants to waters of the United States through the National Pollutant Discharge Elimination System (NPDES) process. The preferred project alternative will require a NPDES Construction General Permit (CGP) for total land disturbance of equal to or greater than one acre, and for stormwater discharges to Waters of the U.S. Pursuant to the requirements of the CGP, the project proponent, or designee, will prepare a Storm Water Pollution Prevention Pollution Plan (SWPPP) to document stormwater control measures during the construction periods for the project. Following completion of the SWPPP, the proponent or designee will complete and submit to EPA a Notice of Intent (NOI) to discharge stormwater. The selected contractor will be responsible for obtaining the NPDES CGP and preparing the SWPPP after award of the contract. There is no review time for a NPDES CGP permit. The electronic NOI (eNOI) has to be submitted at least 14 days prior to the start of construction.

Potential State Permitting and Approval Requirements

Massachusetts Environmental Policy Act

The Massachusetts Environmental Policy Act (MEPA) applies to projects in Massachusetts that exceed defined thresholds and involve state agency action (i.e., projects that are either proposed by a state agency or require a permit, financial assistance, and/or land transfer from one or more state agencies). Projects that fall within MEPA jurisdiction are generally reviewed in a two-step process, beginning with the filing of an Environmental Notification Form (ENF), followed by an Environmental Impact Report (EIR) if needed. As it is currently designed, the preferred alternative does not exceed any MEPA thresholds outlined in the MEPA Regulations (301 CMR 11.00), therefore an ENF would not be required.

Massachusetts Department of Environmental Protection - Chapter 91 Waterways License

The Massachusetts Waterways Regulations administer the provisions of MGL c. 91, the Massachusetts Public Waterfront Act. Chapter 91 preserves the rights of the public to have access to tidelands and waterways of the Commonwealth, and regulates activities on both coastal and inland waterways. The proposed sewer project does not propose work within filled tidelands or within waterways and therefore will not require a Chapter 91 Waterways License.

Massachusetts Department of Fish and Game - Natural Heritage and Endangered Species Program

Aucoot Cove and its surrounding area is designated as Estimated Habitat of Rare Species and Priority Habitat of Rare Species. As a part of the Marion and/or Mattapoisett Conservation Commission NOI permitting process detailed below, coordination with the NHESP will be required to ensure that the project will not result in a “take” on a state listed threatened or endangered species. The NOIs submitted to the Marion and/or Mattapoisett Conservation Commission can be reviewed jointly with NHESP. According to 321 CMR 10.14 (6), construction of new sewer lines within paved roadway is an exempt activity and does not require NHESP review.

Massachusetts Office of Coastal Zone Management (CZM) - Coastal Zone Management Policy Guide

CZM is the policy and planning agency for coastal and ocean issues. Any project located within CZM jurisdiction which requires a federal permit or is considered a federal action requires federal consistency review with the CZM Policy Guide. This consistency review will occur during the USACE PCN process, if required, or as an independent action. This coordination takes approximately three months.

Massachusetts Historical Commission

Section 106 of the National Historic Preservation Act (NHPA) requires that project areas be evaluated to determine the presence of cultural resources. Any new construction projects or renovations to existing buildings or structures that require state funds, licenses, or permits are subject to the review requirements of the MGL Chapter 9, Sections 26-27c, as amended by Chapter 254 of the Acts of 1988 (950 CMR 71.00). As a part of the SRF process, a Project Notification Form (PNF) will be submitted to the Massachusetts Historical Commission to determine if the project will affect any significant cultural or archaeological resources. This coordination takes approximately one month.

Tribal Historic Preservation Officers (THPO)

As a part of USACE permitting the THPO's for the Wampanoag Tribe of Gay Head (Aquinnah), and Mashpee Wampanoag Tribe will be contacted to determine if the project will affect any significant tribal cultural or archaeological resources. This coordination takes approximately one month.

Massachusetts Department of Transportation

Massachusetts Department of Transportation (MassDOT) regulates work within State-owned roadways. Mill Street (Route 6), is owned by MassDOT and therefore work within this road will require the procurement of a MassDOT Access permit to complete the proposed work. This coordination can take approximately one month to prepare and one to three months to obtain the permit.

Potential Local Permitting Requirements

Marion Conservation Commission

The Marion Conservation Commission regulates all proposed work within and adjacent to wetland resource areas within Marion subject to jurisdiction under the Wetlands Protection Act (WPA) and the Town of Marion's Local Wetland Protection Standards. The proposed new sewer mains and associated work is anticipated to take place within the following regulated wetland resources areas: Land Subject to Coastal Storm Flowage (LSCSF), Inland Bank, potentially BVW and/or Salt Marsh, and the 200-foot Riverfront Area to Aucoot Creek. Due to the route of the preferred alternative in both Marion and Mattapoisett, a joint filing of a NOI with both the Marion Conservation Commission and the Mattapoisett Conservation Commission is required. The joint filing approach would need to be previously agreed upon by both Conservation Commissions. If both Conservation Commissions do not approve of the approach to file jointly, then separate permit applications will be filed with each Conservation Commission. This permitting process takes two to three months.

Mattapoisett Conservation Commission

The Mattapoisett Conservation Commission regulates all proposed work within and adjacent to wetland resource areas in Mattapoisett subject to jurisdiction under the WPA and the Town of Mattapoisett's Conservation Commission By-law. The proposed new sewer mains and associated work is anticipated to take place within LSCSF. Due to the route of the preferred alternative in both Marion and Mattapoisett, a joint filing of a NOI with both the Marion Conservation Commission and the Mattapoisett Conservation Commission is required. The joint filing approach would need to be previously agreed upon by both Conservation Commissions. If both Conservation Commissions do not approve of the approach to file jointly, then separate permit applications will be filed with each Conservation Commission. This permitting process takes two to three months.

Intermunicipal Agreement Considerations

Connection of Mattapoisett properties to the Marion collection system would require execution of an intermunicipal agreement (IMA) between the two towns. The IMA would need to address a number of issues, including the following:

- Sewer construction cost sharing
- Future WWTP upgrade cost sharing
- WWTP O&M cost sharing
- NPDES co-permittee issues
- Sewer billing/metering
- Sewer use regulations
- Infiltration/inflow

These issues would need to be evaluated further if this project were to move forward and addressed in the context of the final collection system configuration, limits and number of properties in each neighborhood connected.

In a recent development on this issue, the Town of Mattapoisett has indicated that they would not be interested in connecting the Harbor Beach neighborhood to Marion's collection system and WWTP. Mattapoisett currently conveys flow from their existing sewer collection system to the Town of Fairhaven for treatment. They have purchased sufficient capacity at the Fairhaven WWTP to serve the Harbor Beach neighborhood (and other adjacent areas) and have developed conceptual plans to extend sewers to the area in the future. However, Mattapoisett has indicated that it would be open to discussing connecting Marion's Indian Cove neighborhood to their collection system if/when it gets extended to serve the Harbor Beach neighborhood. That is, under this scenario the flow from a future collection system for the neighborhoods in the project area would be conveyed into Mattapoisett and ultimately to the Fairhaven WWTP instead of into Marion and its WWTP as originally envisioned. Marion has indicated a willingness to consider and potentially discuss this alternative solution moving forward. This alternative may be advantageous to Marion as it would maintain capacity at its WWTP to connect other areas in Town to the collection system.

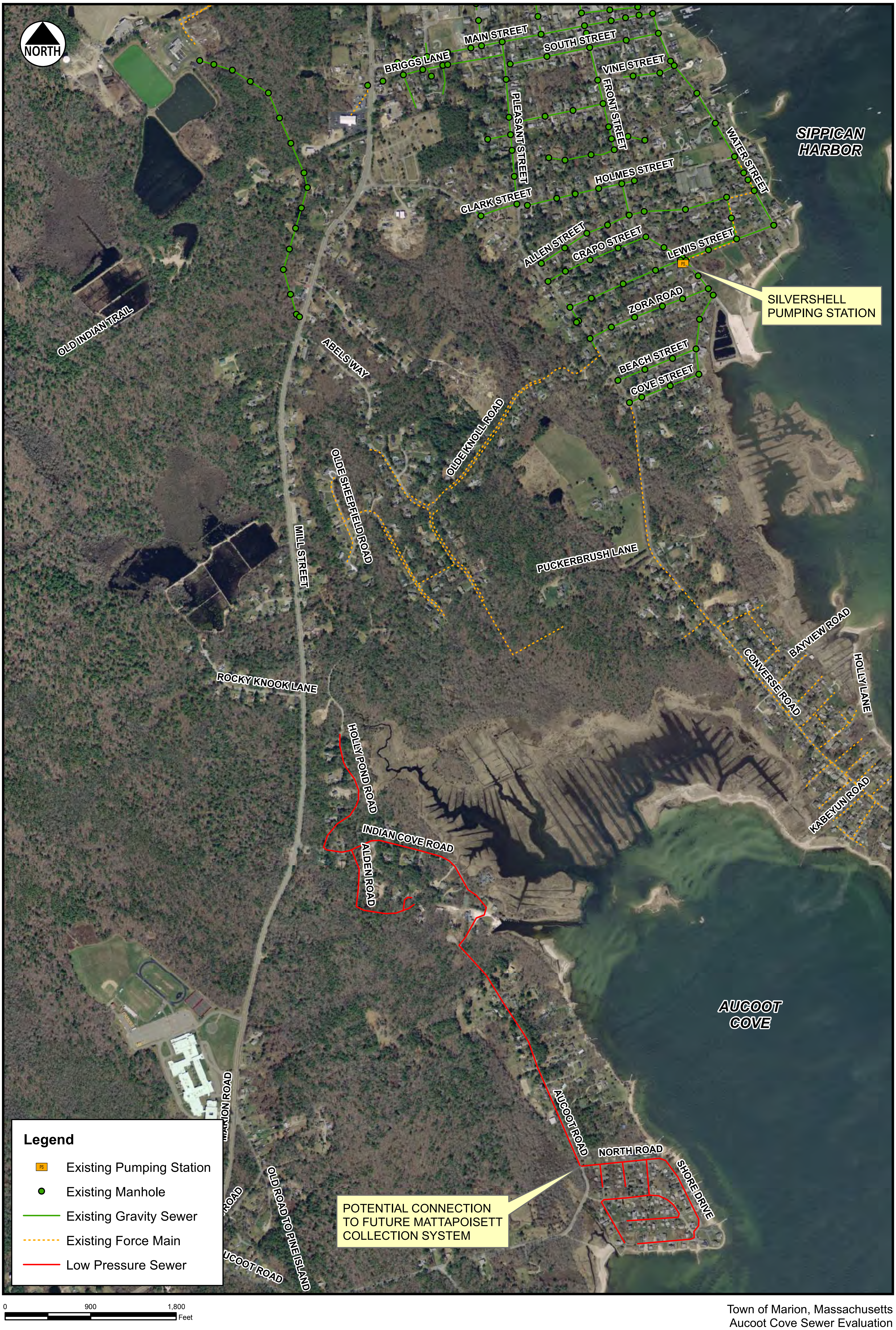
The majority of the evaluation performed as presented in this memorandum would still be valid for this alternative solution. Whether the ultimate discharge location for sewers in these neighborhoods is the Marion or Mattapoisett collection system, this evaluation and recommendation of the most appropriate type of system to serve the Indian Cove and Harbor Beach neighborhoods (pressure sewers) would still apply as would the design drawings developed (with the exception of the portion of the proposed system in Route 6).

This alternative solution would consist of low pressure sewers extending from Holly Pond Road in Marion to Shore Drive in Mattapoisett with the pipelines in the same locations on the neighborhood roads as shown on the design drawings in Attachment B. The pressure sewer in Route 6 would be eliminated, and the system would pump flow in the opposite direction of the recommended Alternative 2. The system would presumably connect to a future extension of the Mattapoisett system in Aucoot Road. The conceptual configuration of this potential alternative solution is shown in **Figure 5**. The opinion of probable construction cost for this alternative solution, for the limit of pipelines shown in Figure 5, is approximately $4.5M. This estimate was developed using comparable publically bid projects and includes pressure sewer main line pipe, services, grinder pumps, provisions for rock/boulder excavation based on the excavation depth, paving, contractor overhead and profit, and contingencies commensurate with the level of design development. The costs are in 2017 dollars and have not been escalated to the mid-point of construction since the implementation timeframe is not clear at this time. This estimate does not include any costs for extending the existing Mattapoisett collection system up to Aucoot Road; it only includes the pipelines shown in Figure 5.

Closing

As discussed above, based on the WWTP capacity analysis included in Attachment A, it would only be feasible to extend sewer service to both neighborhoods in the project area if Marion decides to extend its outfall to the head of the salt marsh in order to eliminate the phosphorus limit included in its new NPDES permit. Otherwise, the WWTP would not have sufficient capacity to service all 158 homes in the project area due to the decrease in available capacity associated with chemical phosphorous removal at the WWTP. There would be sufficient capacity to provide service to the Indian Cove neighborhood without extension of the outfall.

Further discussions will need to occur between Marion and Mattapoisett to explore the possibility of sending flow to Mattapoisett instead of to Marion. This development occurred too late in this evaluation process to pursue this scenario further.



Town of Marion, Massachusetts
Aucoot Cove Sewer Evaluation

CDM Smith

Figure 5
Low Pressure Sewer

Disclaimer

This project has been funded wholly or in part by the United States Environmental Protection Agency under assistance agreement CE-96198501 to the Massachusetts Executive Office of Energy and Environmental Affairs Buzzards Bay National Estuary Program. The contents of this document do not necessarily reflect the views and policies of the Environmental Protection Agency, nor does EPA endorse trade names or recommend the use of commercial products mentioned in this document.

cc: Joseph Costa, BBNEP
Sarah Williams, BBNEP
David Janik, CZM
Todd Callaghan, CZM
Dave Burns, MADEP
Michael Gagne, Town of Mattapoisett
Korrin Petersen, BBC
Norm Hills, Town of Marion
Judy Mooney, Town of Marion
Frank Cooper, Town of Marion
Henri Renauld, Town of Mattapoisett
Shawn Syde, CDM Smith

ATTACHMENT A

WWTP CAPACITY EVALUATION

Memorandum

To: *Town of Marion, MA*

From: *CDM Smith*

Date: *July 12, 2017*

Subject: *Marion WWTP Capacity Evaluation*

1.0 Introduction

The Marion wastewater treatment plant (WWTP) was last upgraded in 2003 – 2005, and was commissioned in September 2005. The current WWTP has now been in operation for more than 11 years. Since startup, the plant has been operated in a consistent, stable manner, with rare exception in compliance with its National Pollutant Discharge Elimination System (NPDES) discharge permit.

The design of the upgraded WWTP was based on flow and load projections that were included in the Town's Comprehensive Wastewater Management Plan (CWMP), which was approved by the Massachusetts Department of Environmental Protection (DEP) in 2002. These projections were made circa 2000-2001, and were based on flow and load data that were available at the time, as well as an accounting of existing and projected new sewer connections that contribute to the plant and per-capita estimates for flow and load. In the 11+ years that the upgraded WWTP has been in operation, the Town has collected a considerable amount of data for reporting and to inform operational decisions at the plant. The resulting database allows for a detailed, updated evaluation of the actual flows and loads the plant has treated and trends related to flows, loads, the operating criteria of the key unit processes, and the overall efficiency of the plant in meeting its permitted discharge criteria.

The purpose of this technical memorandum is to evaluate the actual influent flows and loads and plant operation, and determine the available remaining treatment capacity of the plant at this time. CDM Smith completed a similar evaluation for the Town shortly after plant startup (*Evaluation of Wastewater Treatment Plant Flows and Loads and Available Capacity, April 2008*). It has been nine years since that earlier evaluation was completed, however, and this updated evaluation is needed so the Town has the information necessary to consider future growth and development opportunities and to plan for possible future increases in WWTP capacity.

2.0 Approach to Evaluation

The first step of this evaluation will be to summarize the design criteria for the upgraded WWTP, considering any relevant modifications to equipment, systems or controls that have been made since initial startup. The design criteria include the influent flows and loads, the required effluent quality and each unit process basis-of-design. Second, the actual operation of the plant from 2008 to present, with more emphasis on recent data, will be summarized.

Comparison of the original design criteria and the actual operating criteria will allow for a determination of available treatment plant capacity. This determination will include discussion of process capacity, hydraulic constraints and key design assumptions. The capacity of the unit processes is impacted by the level of treatment that needs to be provided, and the Town now has new limits on total phosphorus and total nitrogen with its renewed NPDES permit. The capacity impacts of these permit limits will be described.

Infiltration and inflow (I/I) can have significant impacts on permit compliance and plant operations; therefore, I/I is discussed in this context.

The Town has historically tracked the number of sewer connections contributing wastewater to the plant, and this memorandum also includes an accounting of these connections. The available plant capacity will be presented in terms of available remaining typical residential connections.

3.0 WWTP Design Criteria

Table 1 summarizes the original design criteria for the upgraded WWTP. The following paragraphs describe key aspects of the design criteria as they relate to the plant's capacity, with significant post-startup modifications noted. Refer to **Appendix A** for a process schematic of the treatment plant.

- The average annual day flow (0.588 mgd) is defined as the total flow treated in a calendar year, divided by the number of days in the year. The average annual day flow is NOT the key criterion with regards to the design of the treatment plant, but is one of the plant's discharge permit limits – if the 12-month rolling average flow exceeds 0.588 mgd, the plant is technically in violation.

- The maximum-day flow rate (WWTP flow-through capacity) of 1.18 mgd is the design peak process capacity of the plant downstream of preliminary treatment. The biological system, effluent filters, and UV disinfection processes are designed to handle a peak flow of 1.18 mgd. It is important to note that in practice, the WWTP has not processed up to 1.18 mgd, primarily a consequence of the SBR control system settings, which are unstable at flow rates approaching 1.18 mgd, and also due to operator discretion. The instrumentation and automatic programming of the sequencing batch reactors (SBR) control system, and the

Design Flows and Loads	
Wastewater Flows (mgd)	
Average Annual Day	0.588
Maximum Day (WWTP flow-through capacity)	1.18
Maximum Day	1.84
Peak Hour	3.00
Influent Loads (lbs/day)	
Average Day	
BOD	909
TSS	880
TKN	191
Lagoon Recycle Loads, May-Oct (lbs/day)	
Average Day	
BOD	83
TSS	neglect
TKN	56
Total Design Loads (lbs/day)	
Average Day	
BOD	992
TSS	880
TKN	247
Maximum Month (summer)	
BOD	1202
TSS	1144
TKN	330
Maximum Month (winter)	
BOD	1036
TSS	1144
TKN	218
Key Effluent Requirements	
BOD	
Average monthly concentration (mg/L)	9
Average monthly load (lbs/day)	42
TSS	
Average monthly concentration (mg/L)	9
Average monthly load (lbs/day)	42
Ammonia-Nitrogen	
Average monthly concentration, May 1-May 31 (mg/L)	2.6
Average monthly concentration, Jun 1-Oct 31 (mg/L)	1.74
Total Nitrogen (not a permit requirement, but target performance)	
Average (mg/L)	7-10
Preliminary Treatment	
Inlet Tanks	
Number of Tanks	2
Dimensions, each (LxWxSWD)	25'x3.75'x7.83'
Volume (gal)	10980

CDM Smith

Table 1
Marion WWTP Design Criteria

Influent Screen	
Type	Mechanical Bar Screen
Number of Units	1
Peak Nominal Hydraulic Rating (mgd)	2.97
Spacing (inches)	0.25
Grit Tank	
Number of units	1
Peak Nominal Hydraulic Rating (mgd)	2.5
Dimensions (ft)	
Diameter	7
Hopper Depth	7.17
Headworks Blowers	
Number of units	2
Type	Rotary PD
Capacity per unit (acfm)	150
Motor Horsepower	10
Lagoon Aeration	
Air Requirements (scfm)	375
Number of blowers	2
Type	Rotary PD
Capacity per unit (acfm)	375
Motor Horsepower	15
No. of Diffusers	
Lagoon 1	52
Lagoon 2	20
Lagoon 3	38
Biological Treatment	
Sequencing Batch Reactors	
Number of Units	2
Dimensions (each unit) (ft)	
Length	99
Width	33
Sidewater depth (ft)	
High Water Level (HWL)	19.9
Low Water Depth (LWL)	15
Total Effective Volume (@LWL) (MG)	0.73
SRT @ max month loadings (days)	10.6
MLSS (@ LWL) (mg/L)	4,700
MLVSS (@LWL) (mg/L)	3,760
Air Requirements (scfm)	
Average	620
Max Day	1150
Main Air Blowers	
Number of units	3
Type	Rotary PD
Capacity per unit (acfm)	660
Motor Horsepower	60

Post-SBR Equalization Tank	
Number of Tanks	1
Dimensions (ft)	
Length	83
Width	15
Side Water Depth (HWL)	10.5
Total Volume (gallons)	97,800
SBR Effluent Pumps	
Number of Pumps	2
Capacity per pump (mgd)	1.2
Motor Size (HP)	7.5
pH Control	
Chemical	Soda Ash (Na_2CO_3)
Dosage (mg/L)	137
Silo Capacity (cf)	576
Silo Dimensions (ft)	
Diameter	12
Height	20
Effluent Filtration	
Filter Type	cloth-disk
Number of Filters	2
Number of disks/filter	2
Filter surface area (sf)	
per disk	54
per filter	108
total	216
Hydraulic Loading Rates	
Average Day (gpm/sf)	1.9
Maximum Day (gpm/sf)	3.8
UV Disinfection	
Type	Low-pressure amalgam
Number of Reactors	1
Number of banks	2
Number of modules/bank	4
No. of lamps/module	6
Number of lamps (total)	48
Sludge Removal	
Waste Activated Sludge Quantity (lbs/day)	
Annual Average	879
Maximum Month (summer)	1,165
Maximum Month (winter)	1,000
WAS Pumps	
Number of Pumps	2
Capacity per pump (gpm)	150
Motor Size (HP)	5

operators' discretion with regard to optimum treatment, currently set the maximum treatment flow rate to about 1 mgd.

- Flow that exceeds the maximum-day capacity of the biological and subsequent treatment plant processes (design of 1.18 mgd, in practice about 1 mgd as discussed above) up to the peak hour influent flow rate (3.0 mgd) is not meant to be treated immediately at that flow rate through the plant. After passing through the preliminary treatment process, flow in excess of 1 mgd is sent to the lagoon system for storage and subsequent full treatment once the influent flow rate subsides. Yard pipe valving also provides the operators with the option of sending excess flow directly to the lagoons, prior to receiving preliminary treatment.

- The peak flow rate to the plant (3.0 mgd) is also the peak (firm) design capacity of the Front Street Pump Station and discharge force main that conveys all the Town's wastewater flow to the plant.

- The maximum-month design loads, and more specifically the BOD load, were the key criteria used to size the biological system. The maximum-month load is defined as the maximum 30-day running average load to the plant. Since the plant must meet its discharge permit on a monthly basis, this represents the maximum load that the plant must be able to handle during any reporting period. Note that the plant was not designed to meet a specific effluent total nitrogen (TN) limit so the influent nitrogen load was not a key design criterion in terms of tankage volume requirements, although it was anticipated that the effluent TN concentration would range from 7 – 10 mg/L under the design loading conditions.

- The key effluent water quality requirements related to the treatment capability of the SBR system, and hence this evaluation, are BOD, TSS, ammonia and TN (and with the new permit, TP). The effluent filters and the UV disinfection system are also key treatment process components but generally are not the limiting unit process with regards to capacity.

- The inlet tanks, influent screen and grit tank are designed based on peak hydraulic requirements (i.e., flow, not load). Therefore, these preliminary treatment processes, and their interconnecting channels, are sized to handle the projected peak hour flow of 3.0 mgd.

- The design of the upgraded plant utilized the previously-existing lagoon system for peak flow equalization. These lagoons have ample storage volume to meet the plant's needs, and do not limit the peak flow handling capability of the plant in any way. The presence of the lagoons is one factor that makes the wastewater load – as opposed to the flow – the key determinant of plant capacity. The lagoons also provide a valuable storage option for the plant, in the event that one of the SBRs is out-of-service for planned or emergency maintenance.

- The SBRs are the key process that enables the plant to meet its discharge permit for BOD, TSS and ammonia, and to remove total nitrogen. As stated above, the SBRs were designed based on the maximum-month loads. Because the biological system performance varies with wastewater temperature, the capacity of the system differs in summer and winter as shown in Table 1. Generally, the SBR process must be designed considering both flow and load, since it functions as both a biological reactor and a clarifier. However, in the case of the Marion WWTP – with the lagoons available for peak flow equalization – the load controls the sizing/capacity of the tanks.

- In order to meet the plant's effluent permit limit for ammonia-nitrogen, and the now the permit limit for effluent TN, the process is designed to provide year-round nitrification (ammonia conversion to nitrate). Selection of the design aerobic solids retention time (SRT) of 10.6 days was one of the key considerations of the design. The design aerobic SRT of 10.6 days provides a process safety factor for nitrification of about 2.5, which is typical practice, in the coldest wastewater temperature condition (10.2 °C).
- The post-SBR equalization tank is needed to dampen the peak discharges from the SBRs during the decant phase. Flow from this tank is pumped to the effluent filters. This pumping system is designed to pump up to the maximum plant throughput (1.18 mgd) with one pump on standby.
- The effluent filters were originally installed to treat lagoon effluent, before the SBR process became operational. Assuming that the SBR process functions properly, the solids load to the filters is low and the capacity of these units is based on hydraulic loading (i.e., flow).
- The UV disinfection system is also sized based on flow, assuming the filtered effluent quality is within anticipated performance, sufficiently high to enable the UV system to provide an adequate dose for disinfection.
- The waste activated sludge (WAS) and filter backwash generated as a product of treatment is pumped to the lagoons for further biological breakdown. Projection of the amount of WAS generated per pound of BOD removed (referred to as "Net Yield") is a key consideration in the sizing and capacity of the SBRs, and this is discussed below.

4.0 Summary of WWTP Operation (2008 – 2016)

The upgraded WWTP has been operational for more than eleven years and sufficient data has been collected to evaluate the actual loading conditions and the unit processes. For this memorandum, data from 2008 to the present were evaluated, with more emphasis attached to the most recent five years of data (August 2011 to present). **Appendix B** includes a series of charts that are useful in assessing the actual operating data. The following paragraphs refer to the charts in Appendix B and address key findings from the data assessment.

- **Figure 1** presents plant influent flows from January 2008 through July 2016. Influent flow is continuously measured and reported daily. Figure 1 shows significant fluctuations, from a high of 1.6 mgd in May 2008 down to about 0.25 mgd in the fall of several years during this period. Figure 1 also shows the results of the 365-day moving average of daily influent flow and the NPDES flow limit. This 365-day moving average was well above 0.6 mgd through mid-2010, but shows a significant reduction in the period since, averaging about 0.50 mgd in the 5-year period from August 2011 through July 2016. As stated above, the permitted 12-month moving average flow is 0.588 mgd, and the plant has been in compliance with this requirement in recent years. The overall reducing trend of average influent flowrate illustrates both the extent of the infiltration/inflow (I/I) issue that exists in Marion, and the success of recent I/I reduction measures implemented by the Town. I/I is further addressed later in this memorandum.

- **Figure 2** presents influent BOD loads from January 2008 through July 2016. Influent BOD concentration has typically been tested and recorded once per week based on 24-hour composite samples. This data shows wide fluctuations with single days ranging from less than 200 lbs/day to more than 1,200 lbs/day. The 30-day monthly average has always been less than the maximum-month design loads (1,036 lbs/day in the winter; 1,202 lbs/day in the summer). The average influent BOD load in the 5-year period from August 2011 through July 2016 was 591 lbs/day. The measured maximum-month BOD load in that 5-year period was 926 lb/day (January 2016).

- **Figure 3** presents influent TSS loads from January 2008 through July 2016. Influent TSS concentration has typically been tested and recorded once per week based on 24-hour composite samples. This data also shows very wide fluctuations with single days ranging from less than 100 lbs/day to more than 2,500 lbs/day. The 30-day monthly average has typically been less than the maximum-month design load (1,144 lbs/day), except for one brief period in September 2009. The average monthly influent TSS load in the 5-year period from August 2011 through July 2016 was 527 lbs/day. The measured maximum-month TSS load in that 5-year period was 1,045 lb/day (March 2013).

- **Figure 4** presents estimated influent TKN loads from January 2008 through July 2016. Influent TKN concentration has typically been tested and recorded only once per month, with some periods of more frequent data collection. To develop a reasonable expectation of TKN loading variability, the data from days on which both influent BOD and TKN were collected were evaluated to establish a relationship between these two parameters. This data set indicates that the influent TKN/BOD ratio averaged 0.21, but varied between the typical range of 0.12 to 0.24, and with influent BOD concentration (the higher the BOD concentration, the lower the TKN/BOD ratio. This relationship can be approximated linearly as follows: *TKN = 0.3027 – BOD x 0.00094*). The data shown in Figure 4 is the result of applying this relationship to the BOD data presented in Figure 2. The 30-day monthly average has been less than the maximum-month TKN design loads (218 lbs/day in the winter, 330 lbs/day in the summer). The average monthly influent TKN load in the 5-year period from

August 2011 through July 2016 was 87 lbs/day. The measured maximum-month TKN load in that 5-year period was 152 lb/day (April 2015).

- **Figure 5** and **Figure 6** present effluent BOD and TSS quality achieved from January 2008 through July 2016, respectively. These data show that the plant's effluent quality has been equal or better than required throughout this extended period. Figure 5 shows that the effluent BOD concentration has typically been in the range of 2-8 mg/L, versus the plant's effluent average monthly permit concentration of 9 mg/L. The data shows a brief period of slightly higher effluent BOD results, in the range of 10 – 12 mg/L, in the fall of 2015. Figure 6 presents effluent TSS concentrations, which have always been very low, approximating 0 mg/L (non-detect) for most of the period due to the highly effective effluent filters.

- **Figure 7** presents effluent ammonia-nitrogen concentrations from January 2008 through July 2016, which are almost always below the seasonal permit limits (average monthly concentration limits of 2.6 mg/L in May and 1.74 mg/L from June through October). An exception to this performance occurred in May 2012, a period during which the ability of the plant to nitrify (to convert ammonia to nitrate) was inhibited, apparently due to a toxicity issue caused by excessive use of root killer in the collection system.

- **Figure 8** shows the effluent TN concentration variation from January 2008 through July 2016. The plant has consistently performed better than the target range of 7-10 mg/L, due to the diligence of the plant operating staff and the lower-than-anticipated influent TKN loads as shown in Figure 4. Until the recent permit renewal, the plant had not had a TN limit, but the ability to remove nitrogen was included for its process and environmental benefits.

- **Figure 9** shows the effluent total phosphorus (TP) concentration variation from January 2008 through July 2016. Effluent TP is typically in the range from 1 to 4 mg/L. Until the recent permit renewal, the plant had not had a TP limit, and is not designed to achieve TP removal. The only TP removal that occurs at the plant is the side effect of WAS production and the result of the highly efficient effluent filters, which remove particulate phosphorus as part of effluent TSS polishing.

- **Figure 10** presents the variation of both total and aerobic SRT from January 2008 through July2016, and shows that fairly consistent control has been maintained over this period. The aerobic SRT has typically ranged between 8-13 days. The average aerobic SRT in the 5-year period from August 2011 through July 2016 was 11.7 days, about 1 day longer than the design aerobic SRT of 10.6 days.

- **Figure 11** presents the variation in mixed liquor suspended solids (MLSS) concentration within each SBR from January 2008 through July 2016. In both SBRs, the MLSS concentration has ranged from 1,000 to 2,000 mg/L, which is well below the design MLSS concentration of 4,700 mg/L (at SBR low water level). This is consistent with the lower-than-design influent BOD loading discussed earlier. The wasting data shown in **Figure 12** is also consistent, as the daily wasting has typically ranged from 400-600 lbs/day of WAS. The average wasting rate in the 5-year period from August 2011 through July 2016 was 435 lbs/day, below the anticipated average wasting rate of 879 lbs/day.

5.0 Available WWTP Capacity

The Marion WWTP's capacity is dependent on the treatment requirements required by the plant's NPDES permit. As stated earlier, the plant was originally designed and has been operated to comply with effluent limits for BOD, TSS and ammonia, among other parameters. The plant is now required to achieve a high level of TN and TP removal. Achieving this level of treatment dictates certain operational approaches that impact plant capacity.

As a result, the plant's available capacity is determined in this section based on two scenarios: 1) the permit that was in effect until the recent renewal; and 2) the new permit that includes TP and TN limits.

5.1 Determination of Available Process Capacity – Previous/Original Permit

The capacity of the Marion WWTP process to meet its previous NPDES permit was dictated by the available tank volume and its ability to successfully accommodate the maximum-month influent BOD load in the winter months. Therefore, determination of available capacity must consider the actual measured BOD load versus the design BOD load. In addition, the original design of the biological process to treat the influent BOD load involved the selection of two key parameters: the aerobic SRT and the net yield. Determination of available capacity must consider these two parameters in light of plant operating data.

Table 2 presents a line-by-line determination of the available WWTP capacity to meet its current NPDES permit in terms of maximum-month influent BOD load. Line 1 indicates the tank volume (at low water level) is 0.73 million gallons. Line 2 indicates that design MLSS concentration is 4,700 mg/L, which dictates the total biological mass within the reactors (Line 3), which is also the solids inventory capacity of the process.

Line 4 indicates the percent of time in the SBR treatment cycle during which the contents are designed to be in aerobic conditions. In order to meet the design nitrogen-removal goal (7 to 10 mg/L TN) under maximum load conditions, this aerobic time was designed to be 38% of the total cycle time. Currently the plant is operating with about 46% of the total cycle under aerobic conditions, which gives a cushion for maintaining nitrification; however, as the design capacity of the process is approached, a reduction back to 38% will likely be required and should be anticipated. Therefore, the lower aerobic fraction of 38% is used in this calculation. Line 5, the mass under aeration, is the product of Lines 3 and 4.

Line	Condition	Value	Comment
1	SBR Volume (@LWL) (million gal)	0.73	Existing two SBR tanks
2	Design MLSS Concentration (mg/L)	4700	
3	Total Biomass (lbs)	28615	
4	Aerobic % of Total Cycle Time	38%	Current plant operation is about 46%; however, to maintain nitrogen removal goals at maximum-month loads, it is estimated that aerobic percentage would need to be reduced to 38%.
5	Biomass Under Aeration (lbs)	10874	
6	Aerobic SRT (days)	10.6	Current plant operation is at 11.7 days. Adequate treatment can be maintained at 10.6 days.
7	TSS (WAS + Eff TSS) Produced (lbs/day)	1026	
8	Net Yield (lbs TSS/ d/BODr)	0.89	Based on plant data from August 2011 - July 2016.
9	BOD Removal Capacity (lbs/day)	1153	
10	Current Max Month BOD Load (lbs/day)	926	
11	Allowable Additional Max Month BOD Load (lbs/day)	227	
12	Max Month:Avg BOD Load Peaking Factor	1.44	Based on plant data (99th percentile of 850/average of 591 = 1.44).
13	Additional BOD load under average conditions (lbs/day)	157	
14	Per capita BOD load, average (lb/d/capita)	0.17	Based on *TR-16, Guides for the Design of Wastewater Treatment Works* per-connection BOD load.
15	Population Equiv. Additional BOD load	926	
16	Population per typical residential connection	2.45	Per 2010 US census.
17	**Typical Residential Connections available**	**378**	Typical residential connection: 2.45 persons. Flows and loads from non-typical connections need to be considered accordingly.
18	Per capita flow (gpd)	65	
19	Avg additional flow (gpd)	60000	

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Table 2
Marion WWTP Determination of Available Capacity - Current NPDES Permit

Line 6 includes the design aerobic SRT of 10.6 days. While the plant currently operates at 11.7 days, the design value of 10.6 days will provide sufficient process safety factor and allows for treatment of more load. Given the aerobic biomass and the aerobic SRT, the amount of WAS solids that can be removed from the system, mostly by wasting but with a small amount in the process effluent, is calculated in Line 7.

Line 8 presents the net yield used in this calculation. During design, the selection of net yield is based on many factors, including the design SRT of the system and the particular characteristics of the wastewater being treated. The actual measured net yield has varied considerably since January 2008; however, in the most recent 5-year period from August 2011 through July 2016, the net yield has been less variable, averaging 0.76 lbs WAS/lb BOD removed over the entire period, with a maximum long-term average of 0.89 lbs WAS/lb BOD removed. This maximum long-term value is reasonable and is suitable for determination of the available capacity of the plant, and results in the BOD removal capacity of 1,153 lbs/day shown in Line 9.

Lines 10 and 11 illustrate that the plant influent BOD load under maximum-month (winter) conditions can increase by (i.e., the plant can successfully treat) 227 lbs/day more than the current condition. In Lines 12 and 13, using a maximum-month to average month BOD load peaking factor of 1.44 (based on the 99th percentile of the 30-day running average of plant data), this translates into an allowable average BOD load increase of 157 lbs/day.

Since August 2011, plant data indicates the average per-capita BOD load has been approximately 0.15 lb/day/person. This per-capita BOD load is on the low end of what would typically be expected - 0.17 lbs/day/person is the value recommended by *TR-16, Guides for the Design of Wastewater Treatment Works* as representing typical conditions. To be conservative, lines 14 and 15 utilize 0.17 lbs/day/person, to equate 157 lbs/day of BOD with a population of 926. The 2010 US Census indicates that the average-per-residence population in the Town of Marion is 2.45. Lines 16 and 17 therefore show that the plant has the capacity to add the equivalent of 378 *typical residential connections* to its current loading. Lines 18 and 19 show that at an estimated per-capital flow rate of 65 gpd, the estimated total average flow from these connections is about 60,000 gpd.

It is important to note, when discussing connections, the above use of the phrase “typical residential connections”, and as described this term is meant to denote a household with the Town’s average per-household population (2.45). Connections that are not “typical” by this definition – that either have a lower or higher estimated flow and load than a typical connection by way of type/size of dwelling – need to be considered accordingly.

5.2 Determination of Available Process Capacity – Modified Permit

The Marion WWTP now has new, seasonal (April – October) discharge limits for TP and TN. The TP limit is both a monthly average concentration limit of 0.2 mg/L and a mass limit of 0.98 lbs/day;

and the TN limit is a seasonal average concentration limit of 4 mg/L and a mass limit of 19.6 lbs/day.

CDM Smith has previously assessed the alternatives available to meet these permit limits at the Marion WWTP. To meet the TP limit, chemical addition of a coagulant (e.g., alum or ferric chloride) to the SBRs, combined with the high level of effluent filtration provided by the existing filters, is anticipated to be the preferred approach. To meet the TN limit of 4 mg/L, compliance is anticipated by optimizing operation of the SBRs, probably with the addition of supplemental carbon. If this SBR optimization does not achieve reliable compliance with a TN limit of 4 mg/L, a new, tertiary denitrification process would be required for long-term compliance with this very low limit.

Implementing these improvements to achieve TP and TN removal will impact the available treatment capacity of the SBRs. These impacts are described below.

Total Phosphorus

Chemical addition is a very effective, reliable method of removing phosphorus. At the Marion WWTP, coagulant would be added primarily to the SBR influent, allowing the mixing and the detention time provided by the SBRs to thoroughly contact the soluble phosphorus in the tank and convert it to solid form. The precipitated phosphorus would then be removed from the tanks as part of the WAS. Any remaining, low amounts of soluble phosphorus in the SBR effluent would be removed via a secondary chemical addition upstream of the effluent filters, with the precipitated phosphorus removed by the filters.

The precipitation of chemical sludge increases solids generation in the SBRs; and since there is a limit on the solids inventory that can be accommodated in the SBRs (Line 3 in Table 2), the chemical sludge reduces the available biological solids inventory. This reduced biological inventory cascades into a reduction in overall treatment capacity of the SBRs. **Table 3** summarizes this impact. Table 3 is a parallel presentation to Table 2, except row 2A has been inserted to include the increase in MLSS concentration that can be expected with coagulant addition (423 mg/L), and row 2B shows the reduced allowable biological MLSS concentration as a result (4,277 mg/L vs. 4,700 mg/L in Table 2). The remainder of the calculation approach is the same as Table 2. Line 17 of Table 3 shows that due to the TP limit, the plant has the capacity to add the equivalent of 205 typical residential connections to its current loading (down from 378 without the TP limit). Lines 18 and 19 show that at a per-capital flow rate of 65 gpd, the estimated total average flow from these connections is about 33,000 gpd (down from 60,000 gpd) without the TP limit.

It should be noted that these impacts are based on these important assumptions: 1) the influent phosphorus concentration is 6 mg/L (there is very little influent P data available, and the available data averages 5.3 mg/L); the influent ortho-P concentration is 50% of total P (i.e., 3 mg/L); and a dose of about 110 mg/L of alum will be required. These values must be confirmed through jar testing at the plant.

Line	Condition	Value	Comment
1	SBR Volume (@LWL) (million gal)	0.73	Existing two SBR tanks
2	Original Design Biological MLSS Concentration (mg/L)	4700	Max month design MLSS
2A	MLSS increased due to chemical sludge (mg/L)	423	Due to alum addition to the SBRs
2B	Reduced allowable biological MLSS concentration (mg/L)	4277	
3	Total Biomass (lbs)	26039	
4	Aerobic % of Total Cycle Time	38%	Current plant operation is about 46%; however, to maintain nitrogen removal goals at maximum-month loads, it is estimated that aerobic percentage would need to be reduced to 38%.
5	Biomass Under Aeration (lbs)	9895	
6	Aerobic SRT (days)	10.6	Current plant operation is at 11.7 days. Adequate treatment can be maintained at 10.6 days.
7	TSS (WAS + Eff TSS) Produced (lbs/day)	933	
8	Net Yield (lbs TSS/ d/BODr)	0.89	Based on plant data from August 2011 - July 2016.
9	BOD Removal Capacity (lbs/day)	1049	
10	Current Max Month BOD Load (lbs/day)	926	
11	Allowable Additional Max Month BOD Load (lbs/day)	123	
12	Max Month:Avg BOD Load Peaking Factor	1.44	Based on plant data (99th percentile of 850/average of 591 = 1.44).
13	Additional BOD load under average conditions (lbs/day)	85	
14	Per capita BOD load (average)	0.17	Based on *TR-16, Guides for the Design of Wastewater Treatment Works* per-connection BOD load.
15	Population Equiv. Additional BOD load	502	
16	Population per typical residential connection	2.45	Per 2010 US census.
17	**Typical Residential Connections available**	**205**	Typical residential connection: 2.45 persons. Flows and loads from non-typical connections need to be considered accordingly.
18	Per capita flow (gpd)	65	
19	Avg additional flow (gpd)	33000	

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Table 3
Marion WWTP Determination of Available Capacity - with new TP Limit

Total Nitrogen

The Marion WWTP currently produces effluent TN in the range of 3-7 mg/L (refer to Figure 8). On an average monthly basis, the plant has typically produced effluent that would comply with the TN limit of 4 mg/L. This effluent TN concentration implies that effluent nitrite and nitrate (NOx-N) is typically in the 1-3 mg/L range. This is very good performance for the SBR process as currently configured. Maintaining this level of treatment as influent loading increases will be challenging, and measures will be needed to optimize operation of the SBRs to improve the reliability and stability of TN removal. To reliably meet the TN limit of 4 mg/L, it will be necessary to consistently produce low effluent NOx-N, which will require that the SBR cycles be modified to include more anoxic time. More anoxic time can only be provided by reducing aerobic time by a corresponding amount. Fortunately, the effluent TN limit is seasonal, and this allows the SBRs to be operated with sufficient aerobic time through the winter (not reducing the current aerobic time) and operate with more anoxic time from April through October, when the water temperature is warmer. The warmer wastewater temperature allows for a seasonal reduction in aerobic SRT while not reducing treatment capacity and while maintaining the same safety factor on nitrification.

Reducing the aerobic cycle time during the permit season as described will NOT reduce treatment capacity of the process, as the warmer wastewater temperature allows for this volume reduction without diminishing nitrification effectiveness. Therefore, the seasonal 4 mg/L TN limit will not decrease the SBR process capacity.

It must be understood that reliably achieving compliance with an effluent average monthly TN concentration limit of 4 mg/L, as flows and loads to the plant increase, may be beyond the capabilities of the SBR process. Effluent nitrogen is present in several forms: 1) ammonia, which can be expected to reliably average about 0.5 mg/L to 0.75 mg/L from a properly designed and well-operated SBR process like the one at the Marion WWTP; 2) particulate organic nitrogen, which typically comprises about 7 percent of the effluent TSS, such that for every 1 mg/L effluent TSS there will be 0.07 mg/L of particulate organic N (and hence can be about 0.14 – 0.21 mg/L from a plant with effluent TSS of 2-3 mg/L); 3) refractory dissolved organic nitrogen (rDON), which cannot be removed by feasible, available technology and is typically slightly less than 1 mg/L; and 4) NOx-N, which is removed by the denitrification process. To achieve an effluent TN of 4 mg/L, the plant must provide denitrification such that the NOx-N is typically less than 2 mg/L. Reliable performance to this low concentration may or may not be reliably achievable with the plant's SBRs. New tertiary denitrification filters may be required to reliably comply with this limit.

In terms of plant capacity, the 4 mg/L TN limit therefore does not have an impact. If/when reliable compliance with a 4 mg/L TN limit becomes not achievable with the SBR process, it will likely be accomplished by addition of the tertiary process, not by modification (and thereby reduction of the capacity) of the existing treatment processes at the plant.

Summary

In summary, the TP limit of 0.2 mg/L will reduce plant capacity as described above and as shown by comparing Tables 2 and 3. The TN limit of 4 mg/L will not limit plant capacity because the limit is seasonal, and if necessary compliance will likely be achieved be construction and operation of a new, tertiary process.

5.3 Plant Impacts Due to Increased Flow

It has been determined that barring the TP limit, the WWTP has the process capacity to handle an additional load equivalent to 378 typical residential connections, and that the corresponding increase in average wastewater flow is 60,000 gpd. With the TP limit, this flow rate increase is reduced to 33,000 gpd.

This increase in flow will have minimal impact on the operation of the plant, because the current flow averages about 0.50 mgd, and with the addition of 60,000 gpd *(33,000 gpd with the TP limit)* the resulting average flow of 0.560 mgd *(0.533 mgd)* will still be less than the design average annual flow of 0.588 mgd. This is in contrast to the pre-2011 time-frame (refer to Figure 1), when the plant's discharge was already in excess of 0.588 mgd due to extraneous flow (I/I), and adding flow would be challenging from a regulatory perspective. The I/I abatement efforts undertaken by the Town, and the resulting reduction of flow to the WWTP, have helped mitigate this potential issue. It should be noted that continued attention to I/I abatement is important to maintain the ability to add connections.

A flow increase of 60,000 gpd *(33,000 gpd)* would have the following impacts on the plant:

- The preliminary treatment processes (the inlet tanks, the influent screen and the grit tank) will not be notably impacted.

- The lagoons, used for influent flow equalization during high peak flows and for storage and treatment of WAS and filter backwash, would be marginally impacted with no TP limit in place. The quantity of WAS flow and filter backwash is roughly proportional to the treated flow rate, so increasing the average flow treated from 0.50 mgd to 0.56 mgd *(0.533 mgd)* will increase these recycle flows by about 10 percent. The lagoon systems have more than ample capacity and this should have only a small impact on operation. Note that the chemical sludge generated to comply with a TP limit (independent of flow) will increase the solids load in the filter backwash and will impact lagoon operation as well documented elsewhere.

- At the operators' discretion, the lagoons also may be used more frequently for peak-flow equalization, although any increase would be marginal and well within the ability of the lagoons to equalize peak flow.

- The SBR system operation will not be significantly impacted by this flow increase. Changes to SBR operation will be required because of the new TP and TN limits, but not because of the addition of up to 60,000 gpd *(33,000 gpd)* of flow.

- The post-SBR equalization tank is sized to dampen the peak SBR decant rate and allow for a maximum withdrawal rate equivalent to 1.18 mgd. The equalization tank would still be sufficient in size to handle an average flow increase of 60,000 gpd *(33,000 gpd)*.

- Increasing the treated average flow from 0.50 mgd to 0.56 mgd *(0.533 mgd)* will increase the loading rate to the effluent filters by about 10 percent, although still well within the allowable loading rate of the filter media. Filter system redundancy is an issue to be considered, however.

- The UV system is designed to disinfect up to 1.2 mgd and meet the plant's effluent coliform limits of 14 MPN/100 ml geometric mean and 43 MPN/100 ml maximum. The installed UV system is very robust and has been able to maintain compliance with these limits with capacity to spare, and 60,000 gpd *(33,000 gpd)* more flow under average conditions is still well within the capacity of the UV system. UV system redundancy and reliability is an issue to be considered, however.

- The capacity of the plant's effluent pipe was increased during the plant upgrade, and tested at up to 1.8 mgd. The pipe will not be impacted by increasing the average flow through the plant by up to 60,000 gpd *(33,000 gpd)*.

In summary, increasing the plant flow by an average of 60,000 gpd *(or 33,000 gpd due to the TP limit)* will have minimal impacts on the plant and will not require capital improvements.

5.4 Permit and Regulatory Considerations

The Marion WWTP operates and discharges effluent under the conditions of its NPDES permit. This permit places conditions on effluent quality and quantity, and any increase in flow must consider whether there are impacts of the permit.

Flow

The plant's permitted discharge is currently limited to a 12-month rolling average of 0.588 mgd. For the last five-plus years the flow has been less than that permitted rate, at 0.50 mgd. Increasing the average annual flow by as much as 60,000 gpd is allowable and would not result in exceedance of this permit criterion.

Effluent Load

The plant's effluent is required to meet effluent BOD, TSS, TN and TP standards for concentration, and also for load, and the load limits are increasingly challenging as flows increase. However, as long as the flow is less than the design flow, the concentration limits govern the necessary plant performance and therefore load limits are not constraints on a flow increase as long as the resulting flow is less than 0.588 mgd.

5.5 Summary

Under the terms of its previous NPDES permit (with no TP or TN limits), the Marion WWTP has the process capacity to handle an additional 227 lbs/day of influent BOD under maximum-month (winter) conditions, and this equates to the equivalent of 378 typical residential sewer connections beyond current loading. The plant operation would be only minimally impacted. The average flow would increase by an estimated 60,000 gpd as a result of adding these connections, and flow rate would still be within the plant's discharge permit.

With the new TP and TN limits included in the Marion WWTP's NPDES permit, the plant's capacity decreases, and has the process capacity to handle an additional 123 lbs/day of influent BOD under maximum-month (winter) conditions. This equates to the equivalent of 205 typical residential sewer connections beyond current loading. The average flow would increase by an estimated 33,000 gpd.

6.0 Sewer Connection Summary

Table 4 presents a summary of existing, reserved and available typical residential sewer connections. Lines 1 and indicate that there was a total of 1,593 and 1,648 sewer connections in service in 2008 and December 2016, respectively. Lines 3 through 8 are an accounting of connections that were approved in 2008 and have been added/approved since 2008, and Line 8 indicates that there are currently (as of the end of 2016) 127 reserved connections (those that have been previously approved by the Town, but have not yet actually connected plus as-yet unconnected residences from the 40-B development).

Line 9 indicates the available additional connections for both the previous NPDES permit conditions (378 connections) and with the new permit's total phosphorus limit (205 connections) carried forward from Tables 2 and 3. Line 10 summarizes the total allowable typical residential connections to the system for the two permit conditions and Line 11 shows that with the previous permit requirements, there are a total of 251 available typical residential connections in terms of treatment plant capacity. With implementation of the new TP limit, there are a total of 78 available typical residential connections in terms of treatment plant capacity. **It is important to remember and note that for this accounting exercise these are assumed to be typical residential connections.** Actual estimated flows and loads from any proposed sewer extension or connection must be considered and compared to the plant's remaining influent flow and load capacity as described earlier in this memorandum.

Line	Description	Current Permit	with New TP Limit
Actual Connections			
1	Total connections as of 2008	1593	1593
2	Total Connections as of December 2016	1648	1648
Reserved or Approved Connections			
3	Reserved/approved Connections in 2008	121	121
4	Approved Sewer Applications from 2008 - 2016, not Including 40B Development	26	26
5	Not-yet Connected 40B Dwellings (all 3-bedroom homes)	35	35
6	Total Reserved/Approved Connections through 2016	182	182
7	Connections since 2008	55	55
8	Total Reserved Connections in 2016	127	127
Allowable Typical Residential Connections (see Tables 2 and 3)			
9	Total Additional WWTP Typical Residential Connections Allowed	378	205
10	Total Typical Residential Connections Allowable to WWTP	2026	1853
"Available" Typical Residential Connections			
11	Total "Available" New Typical Residential Connections	**251**	**78**

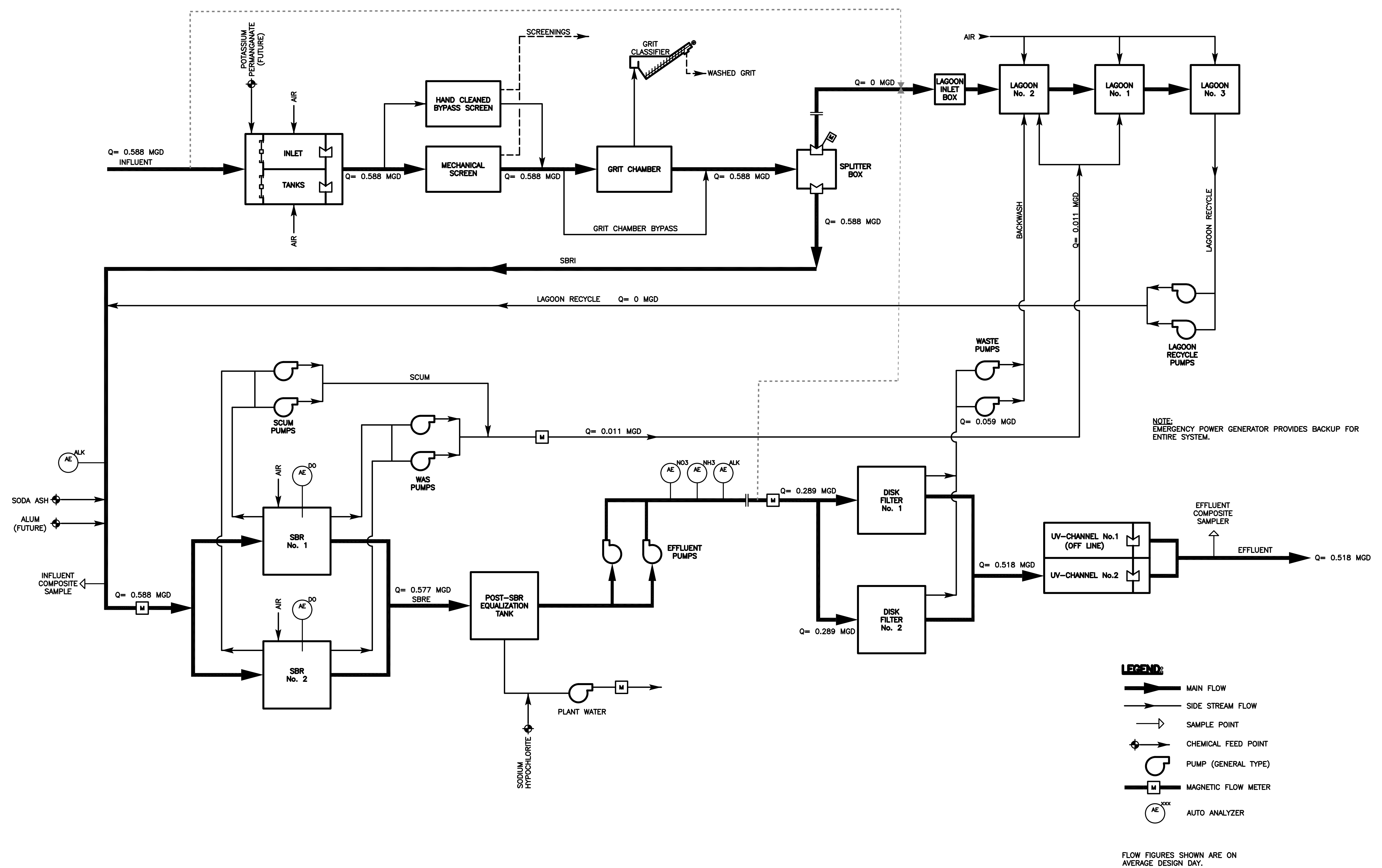
Table 4
Marion WWTP Determination of
Available Typical Residential Connections

8.0 Summary of Conclusions

The evaluations described in this memorandum indicate that:

- Under the previous NPDES permit treatment requirements, the plant has the capacity to add the equivalent of 378 typical residential connections to its current loading. At the per-capital flow rate of 65 gpd, the estimated total average flow from these connections is about 60,000 gpd.
- Under the new NPDES permit treatment requirement to remove phosphorus, the plant has the capacity to add the equivalent of 205 typical residential connections to its current loading. At the per-capital flow rate of 65 gpd, the estimated total average flow from these connections is about 33,000 gpd.
- The plant has a total capacity to handle 2,026 typical residential sewer connections under the previous NPDES permit treatment requirements and 1,853 typical residential sewer connections under the new NPDES permit treatment requirement to remove phosphorus.
- The plant's permitted discharge is currently limited to a 12-month rolling average of 0.588 mgd. Since August 2011, the effluent flow rate has averaged about 0.50 mgd. Adding up to 60,000 gpd to the discharge flow rate would be within the plant's permissible discharge rate. This is a benefit of the Town's ability to achieve successful flow reduction through the I/I removal program.
- Continued implementation of a successful I/I removal program would provide significant benefit to the Town in terms of maintaining available flow capacity in accordance with the NPDES permit and reducing O&M cost.

APPENDIX A: PLANT FLOW SCHEMATIC



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TOWN OF MARION, MASSACHUSETTS
WASTEWATER TREATMENT PLANT UPGRADE

PROCESS FLOW DIAGRAM

FIGURE 1

APPENDIX B: DATA CHARTS

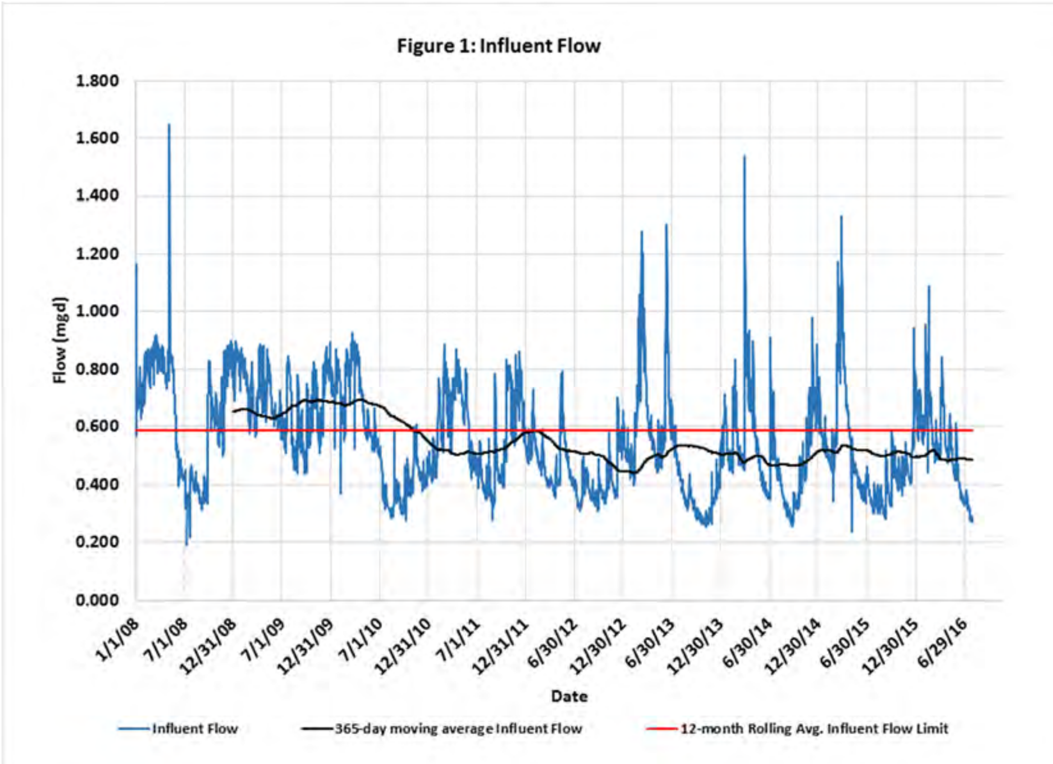
Figure 1: Influent Flow
1.800
1.600
1.400
1.200
1.000
0.800
0.600
0.400
0.200
0.000
Flow (mgd)
1/1/08
7/1/08
12/31/08
7/1/09
12/31/09
7/1/10
12/31/10
7/1/11
12/31/11
6/30/12
12/30/12
6/30/13
12/30/13
6/30/14
12/30/14
6/30/15
12/30/15
6/29/16
Date
Influent Flow
365-day moving average Influent Flow
12-month Rolling Avg. Influent Flow Limit

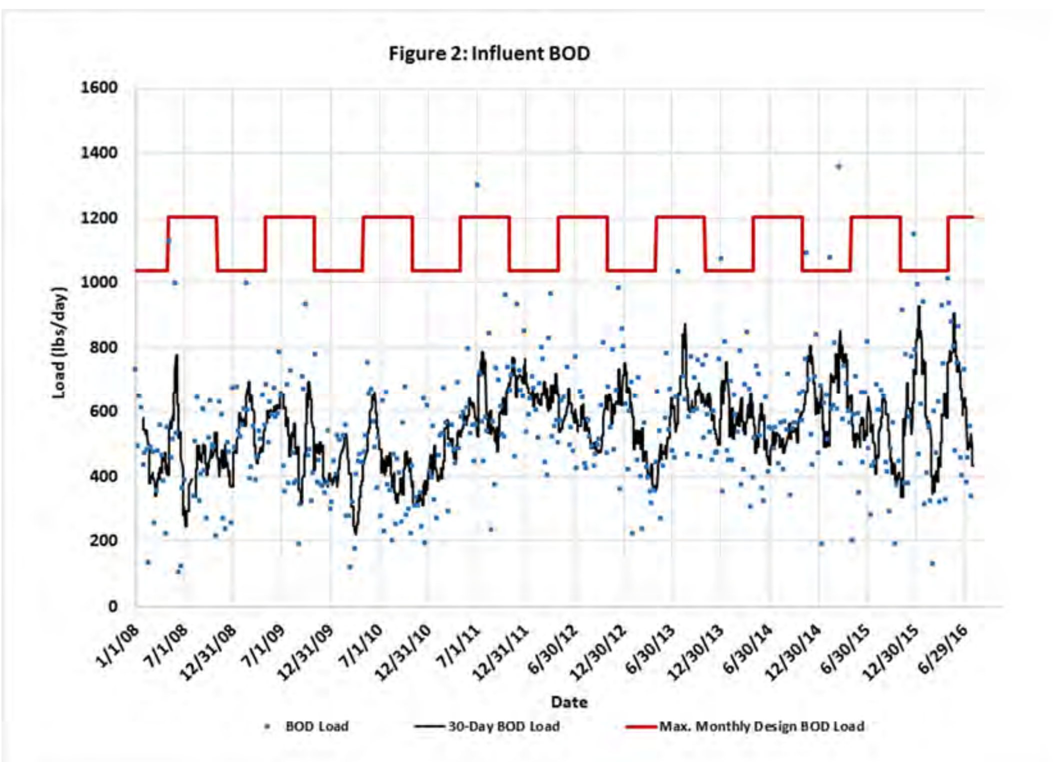
Figure 2: Influent BOD
1600
1400
1200
1000
800
600
400
200
0
Load (lbs/day)
1/1/08
7/1/08
12/31/08
7/1/09
12/31/09
7/1/10
12/31/10
7/1/11
12/31/11
6/30/12
12/30/12
6/30/13
12/30/13
6/30/14
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6/29/16
Date
BOD Load
30-Day BOD Load
Max. Monthly Design BOD Load

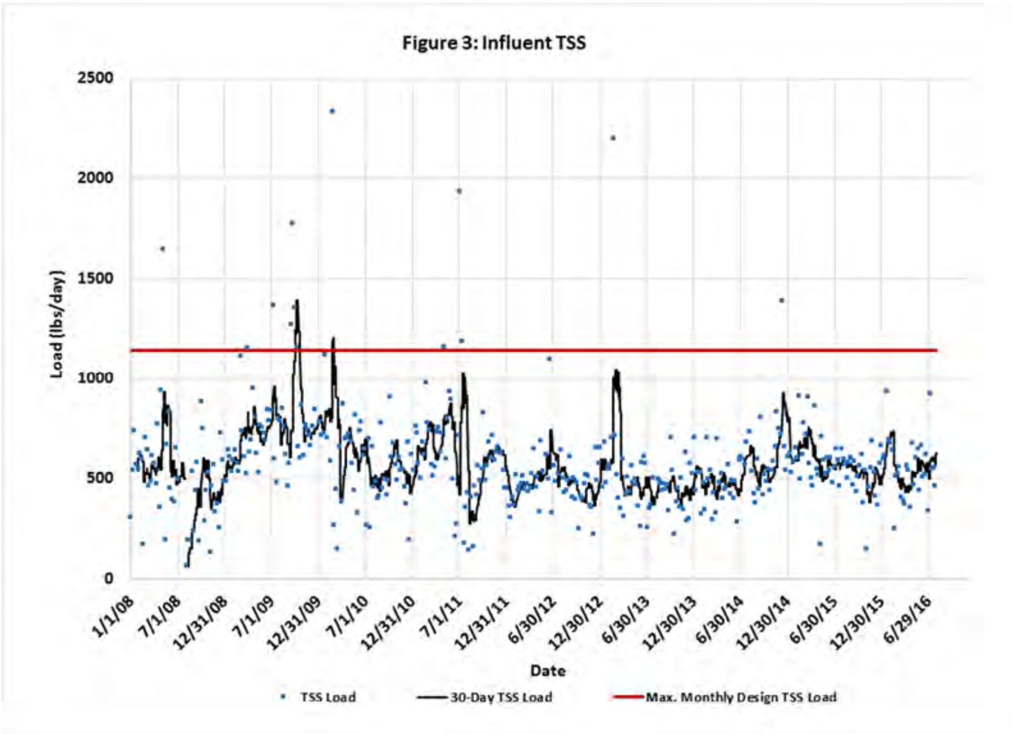
Figure 3: Influent TSS
2500
2000
1500
1000
500
0
Load (lbs/day)
1/1/08
7/1/08
12/31/08
7/1/09
12/31/09
7/1/10
12/31/10
7/1/11
12/31/11
6/30/12
12/30/12
6/30/13
12/30/13
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6/29/16
Date
TSS Load
30-Day TSS Load
Max. Monthly Design TSS Load

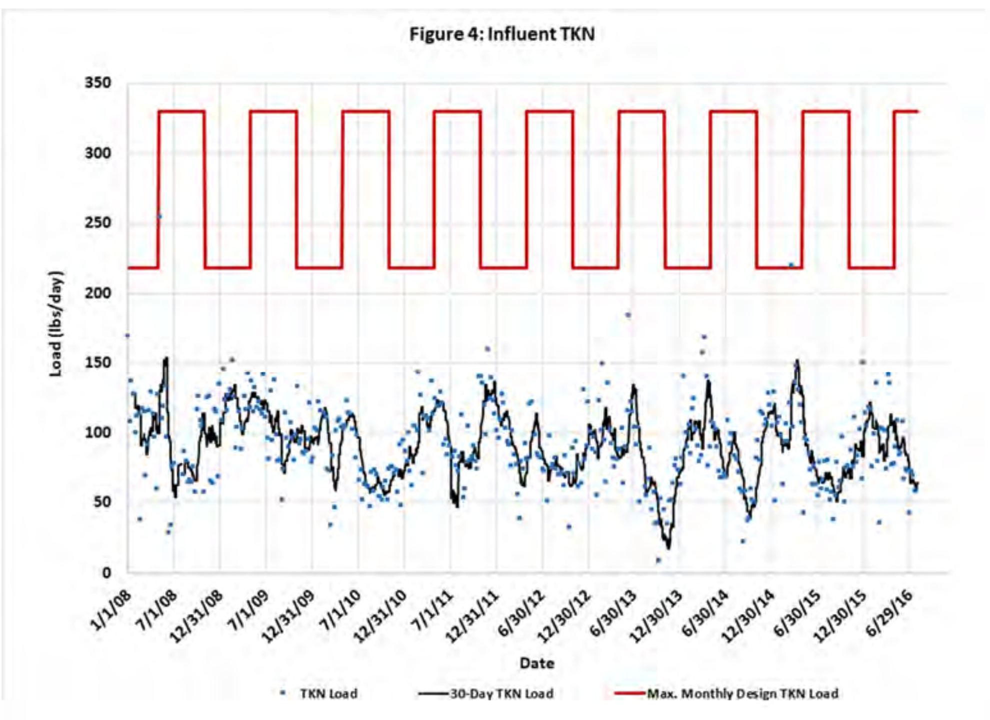


Figure 4: Influent TKN
350
300
250
200
150
100
50
0
Load (lbs/day)
1/1/08
7/1/08
12/31/08
7/1/09
12/31/09
7/1/10
12/31/10
7/1/11
12/31/11
6/30/12
12/30/12
6/30/13
12/30/13
6/30/14
12/30/14
6/30/15
12/30/15
6/29/16
Date
TKN Load
30-Day TKN Load
Max. Monthly Design TKN Load

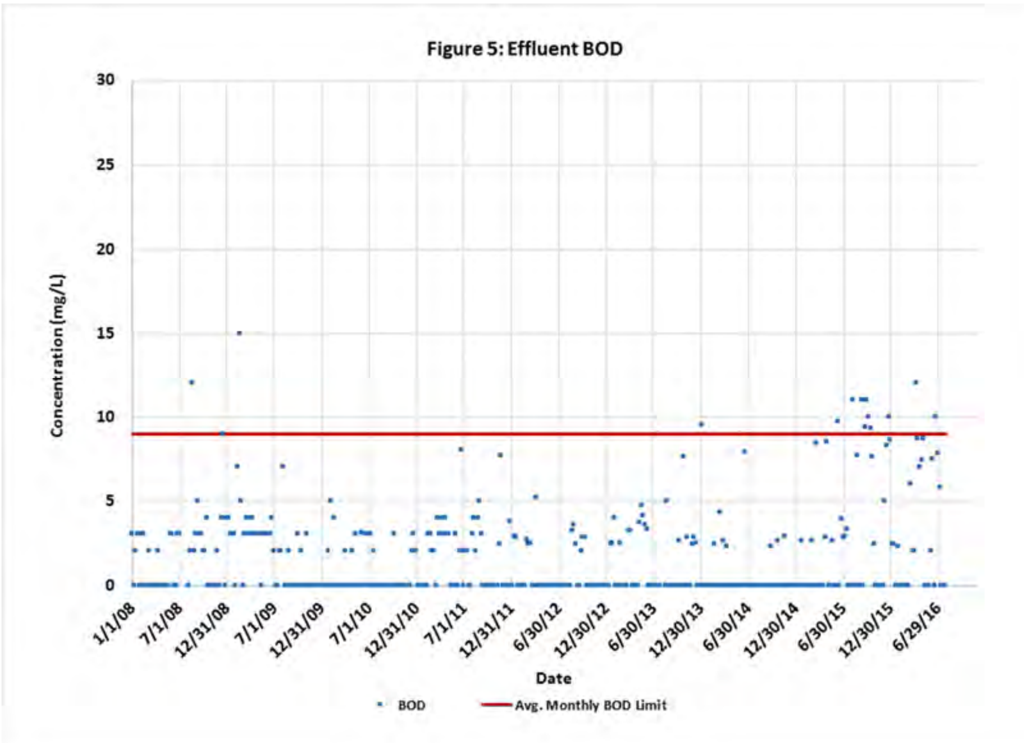


Figure 5: Effluent BOD
Concentration (mg/L)
30
25
20
15
10
5
0
1/1/08
7/1/08
12/31/08
7/1/09
12/31/09
7/1/10
12/31/10
7/1/11
12/31/11
6/30/12
12/30/12
6/30/13
12/30/13
6/30/14
12/30/14
6/30/15
12/30/15
6/29/16
Date
BOD
Avg. Monthly BOD Limit

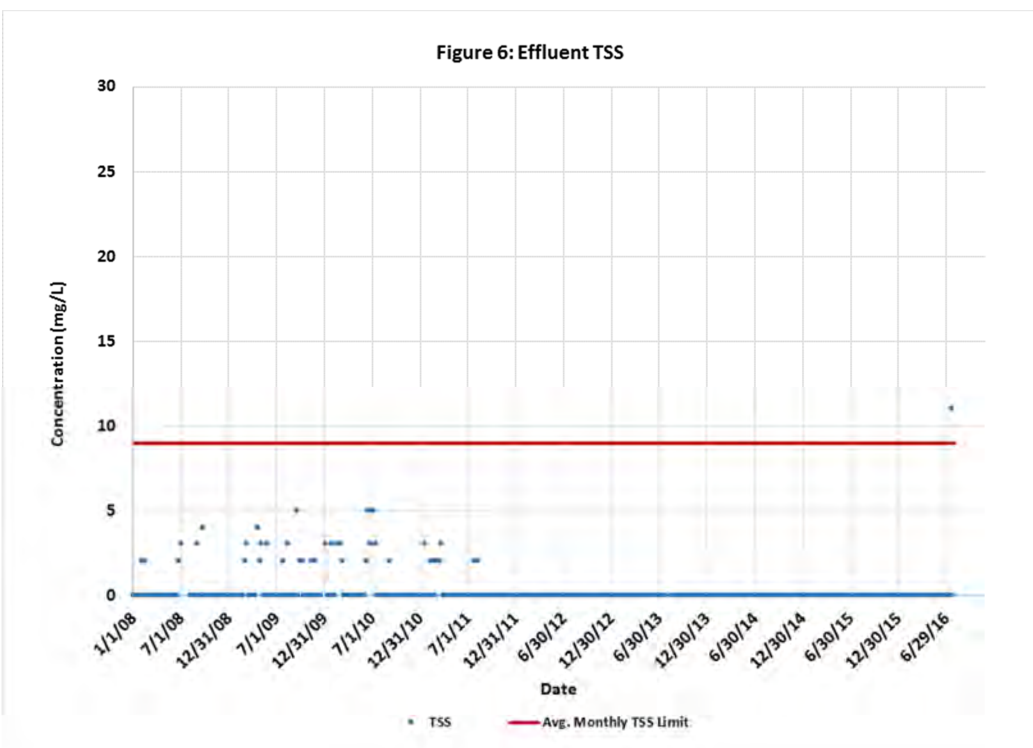


Figure 6: Effluent TSS
Concentration (mg/L)
30
25
20
15
10
5
0
1/1/08
7/1/08
12/31/08
7/1/09
12/31/09
7/1/10
12/31/10
7/1/11
12/31/11
6/30/12
12/30/12
6/30/13
12/30/13
6/30/14
12/30/14
6/30/15
12/30/15
6/29/16
Date
TSS
Avg. Monthly TSS Limit

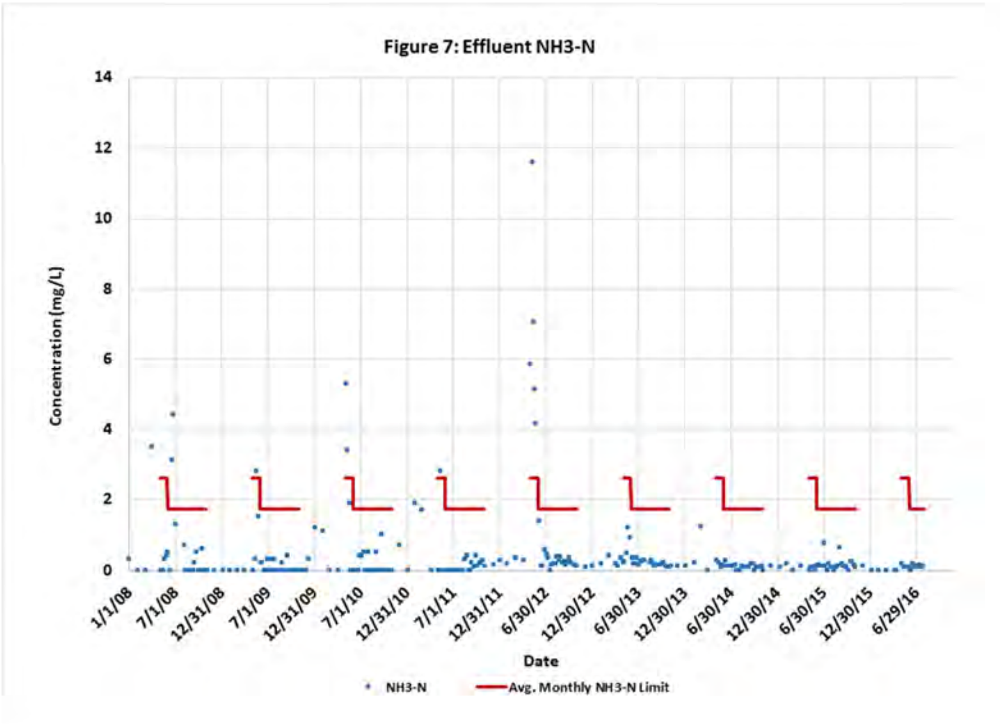


Figure 7: Effluent NH3-N
14
12
10
8
6
4
2
0
Concentration (mg/L)
1/1/08
7/1/08
12/31/08
7/1/09
12/31/09
7/1/10
12/31/10
7/1/11
12/31/11
6/30/12
12/30/12
6/30/13
12/30/13
6/30/14
12/30/14
6/30/15
12/30/15
6/29/16
Date
NH3-N
Avg. Monthly NH3-N Limit

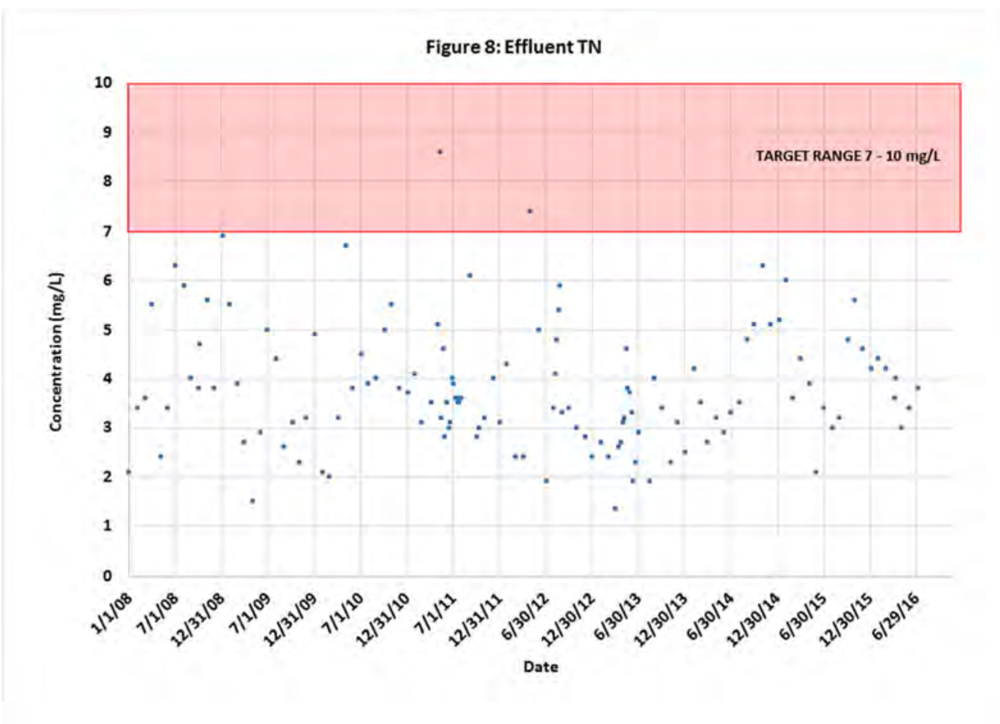


Figure 8: Effluent TN
10
9
8
7
6
5
4
3
2
1
0
Concentration (mg/L)
TARGET RANGE 7 - 10 mg/L
1/1/08
7/1/08
12/31/08
7/1/09
12/31/09
7/1/10
12/31/10
7/1/11
12/31/11
6/30/12
12/30/12
6/30/13
12/30/13
6/30/14
12/30/14
6/30/15
12/30/15
6/29/16
Date

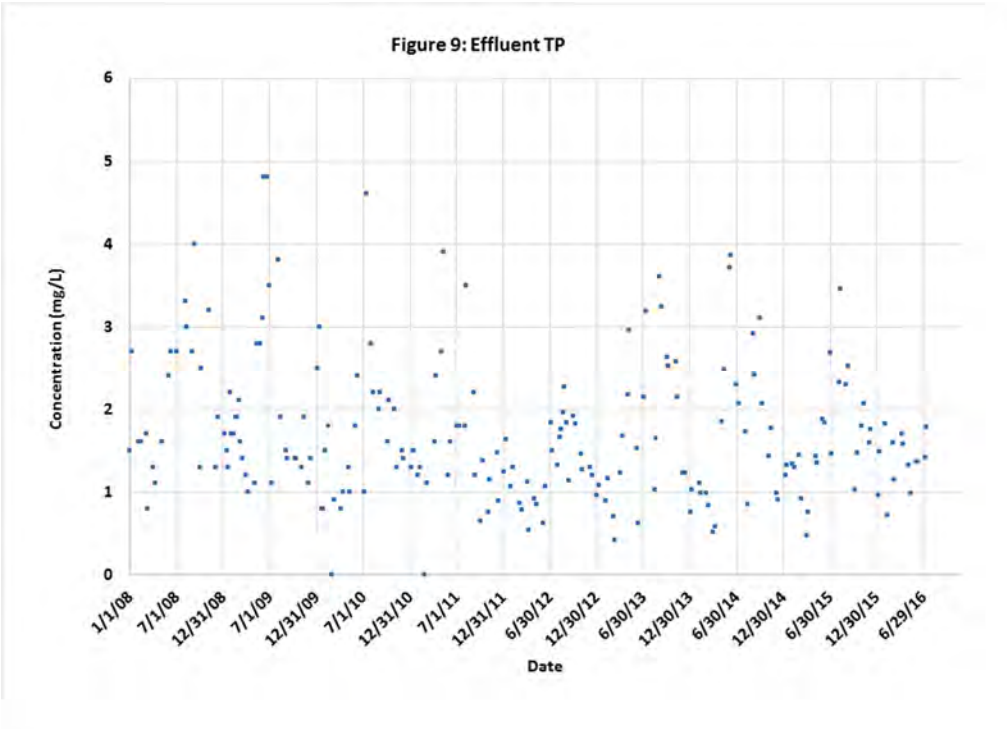


Figure 9: Effluent TP
Concentration (mg/L)
Date
0
1
2
3
4
5
6
1/1/08
7/1/08
12/31/08
7/1/09
12/31/09
7/1/10
12/31/10
7/1/11
12/31/11
6/30/12
12/30/12
6/30/13
12/30/13
6/30/14
12/30/14
6/30/15
12/30/15
6/29/16

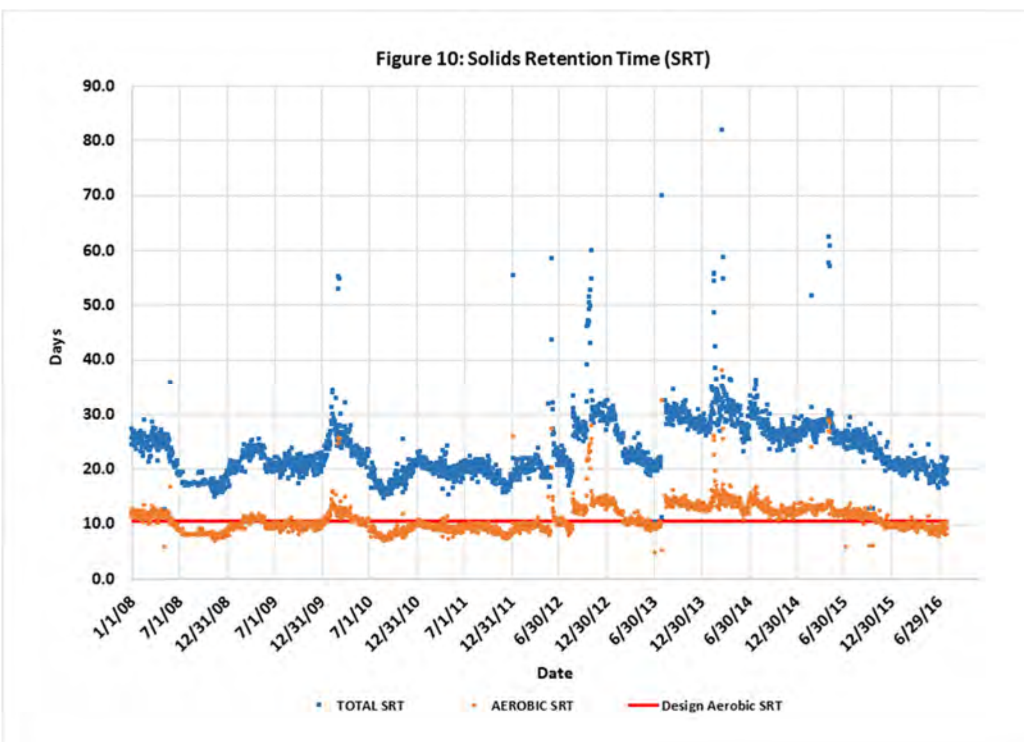


Figure 10: Solids Retention Time (SRT)
Days
Date
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
1/1/08
7/1/08
12/31/08
7/1/09
12/31/09
7/1/10
12/31/10
7/1/11
12/31/11
6/30/12
12/30/12
6/30/13
12/30/13
6/30/14
12/30/14
6/30/15
12/30/15
6/29/16
TOTAL SRT
AEROBIC SRT
Design Aerobic SRT

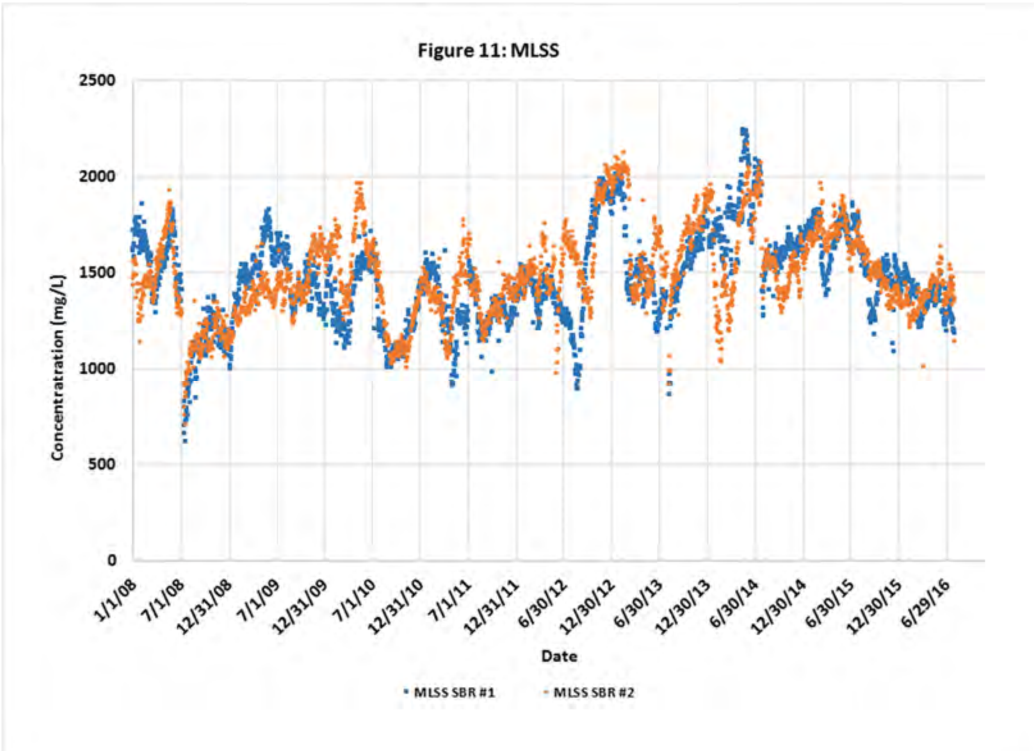


Figure 11: MLSS
Concentratration (mg/L)
2500
2000
1500
1000
500
0
1/1/08
7/1/08
12/31/08
7/1/09
12/31/09
7/1/10
12/31/10
7/1/11
12/31/11
6/30/12
12/30/12
6/30/13
12/30/13
6/30/14
12/30/14
6/30/15
12/30/15
6/29/16
Date
MLSS SBR #1
MLSS SBR #2

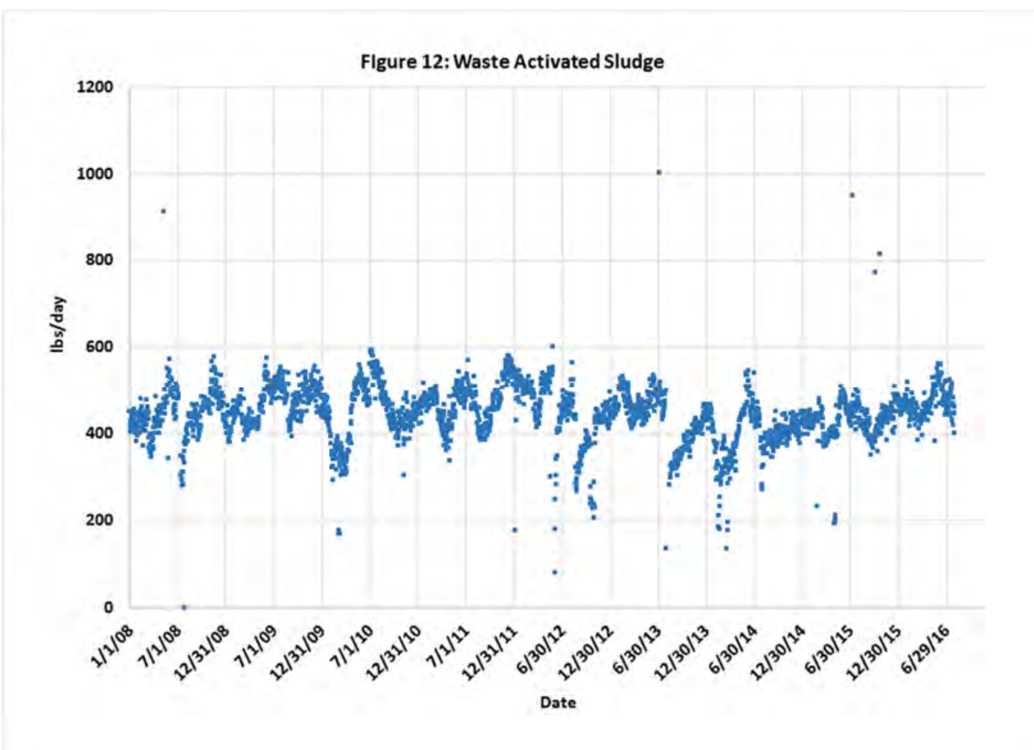


Figure 12: Waste Activated Sludge
lbs/day
1200
1000
800
600
400
200
0
1/1/08
7/1/08
12/31/08
7/1/09
12/31/09
7/1/10
12/31/10
7/1/11
12/31/11
6/30/12
12/30/12
6/30/13
12/30/13
6/30/14
12/30/14
6/30/15
12/30/15
6/29/16
Date

ATTACHMENT B

DESIGN PLANS

TOWN OF MARION, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

BOARD OF SELECTMEN

JONATHAN E. DICKERSON, CHAIRMAN

STEPHEN C. GONSALVES, VICE-CHAIRMAN

NORMAN A. HILLS, CLERK

TOWN ADMINISTRATOR

PAUL F. DAWSON

DEPARTMENT OF PUBLIC WORKS

ROBERT ZORA, SUPERINTENDENT

FRANK COOPER, WWTP SUPERINTENDENT

JULY 2017

LOCATION PLAN

LIST OF DRAWINGS

CDM Smith

PROVIDENCE, RHODE ISLAND

Water ***Environment*** ***Transportation*** ***Energy*** ***Facilities***

0357–114884

GENERAL NOTES

1. PLAN, TOPOGRAPHY, AND UTILITY SURVEY PROVIDED BY SURVEYING AND MAPPING CONSULTANTS (SMC) INC., BRAINTREE, MASSACHUSETTS BASED ON AERIAL PHOTOGRAPHY AND FIELD SURVEY – 2016.
2. CONTRACTOR SHALL OBTAIN ALL PERMITS AND APPROVALS REQUIRED TO PERFORM THE PROPOSED WORK UNDER THIS CONTRACT. CONTRACTOR SHALL CONFORM WITH ANY REQUIREMENTS AND CONDITIONS OF THE TOWN OF MARION AND TOWN OF MATTAPOISETT CONSERVATION COMMISSIONS AND MASSACHUSETTS DEPARTMENT OF TRANSPORTATION.
3. LOCATION OF EXISTING UTILITIES AND SUB-SURFACE STRUCTURES ARE FROM UTILITY SURVEY, TOWN OF MARION AND TOWN OF MATTAPOISETT RECORDS, AND RECORDS FROM PUBLIC UTILITIES. INFORMATION IS CONSIDERED APPROXIMATE, BOTH AS TO SIZE AND LOCATIONS, AND IS SHOWN ON THESE DRAWINGS TO GIVE BIDDERS A GENERAL IDEA OF EXISTING CONDITIONS. IT IS UNDERSTOOD AND AGREED THAT THE CONTRACTOR SHALL NOT RELY UPON THESE DRAWINGS FOR SUCH INFORMATION, BUT SHALL MAKE EXAMINATIONS IN THE FIELD BY VARIOUS AVAILABLE METHODS, AND SHALL OBTAIN INFORMATION FROM PUBLIC UTILITIES AS TO THE LOCATION OF ALL SUB-SURFACE UTILITIES. CONTRACTOR SHALL CALL DIG SAFE® (811) BEFORE EXCAVATION.
4. ELEVATIONS FOR EXISTING STRUCTURES MAY NOT BE SHOWN. CONTRACTOR SHALL OPEN STRUCTURES, PERFORM TEST PITS, OR OTHER INVESTIGATIONS TO CONFIRM LOCATIONS AND CLEARANCE.
5. GAS MAINS ARE ASSUMED TO HAVE THREE FEET OF COVER UNLESS SHOWN OTHERWISE. WATER MAINS ARE ASSUMED TO HAVE FIVE FEET OF COVER UNLESS SHOWN OTHERWISE. IT IS NOT WARRANTED THAT ALL UTILITIES ARE SHOWN. UTILITIES TO INDIVIDUAL BUILDINGS MAY NOT BE SHOWN.
6. NOTES AND INSTRUCTIONS TO CONTRACTORS FOR ALL NEW WORK ARE SHOWN THUS:
7. HOUSE NUMBERS LEFT BLANK COULD NOT BE DETERMINED.
8. CONTRACTOR SHALL NOT DISTURB OR OCCUPY ANY LAND OUTSIDE THE LIMITS OF TOWN OF MARION AND TOWN OF MATTAPOISETT RIGHT-OF-WAYS AND EASEMENTS SHOWN.
9. CONTRACTOR SHALL HAVE SUFFICIENT TRAFFIC CONTROL DEVICES AVAILABLE AT ALL TIMES DURING CONSTRUCTION. SUCH DEVICES INCLUDE VARIABLE MESSAGE BOARDS, LIGHTED ARROW BOARDS, REFLECTIVE BARRELS WITH FLASHING LIGHTS, REFLECTIVE CONES, DETOUR SIGNS, ROAD CLOSED SIGNS, OR OTHER SIGNS AND DEVICES WARRANTED BY THE WORK. CONTRACTOR SHALL PLACE TRAFFIC CONTROL DEVICES AS DIRECTED BY OWNER AND ENGINEER, AND MAINTAIN THEM AS REQUIRED THROUGHOUT CONSTRUCTION.
10. STEEL PLATING SHALL BE PROVIDED AND SECURELY INSTALLED OVER THE TRENCH FOR MAINTENANCE OF TRAFFIC WHEN WORK IS NOT IN PROGRESS.
11. LOCATION OF GRINDER PUMPS TO BE DETERMINED BASED ON FIELD CONDITIONS.
12. VERTICAL DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929 (NGVD 29).
13. PROTECT EXISTING WATER SERVICES DURING INSTALLATION OF THE NEW SEWER SERVICE AND GRINDER PUMP. IF NECESSARY, RELOCATE EXISTING WATER SERVICE AS SHOWN ON DETAIL SHEET NO. CD-1 OR AS DIRECTED BY THE ENGINEER.

LEGEND

NEW

- PRESSURE SEWER
- REDUCER
- BEND
- PRESSURE SEWER AND SERVICE CONNECTION
- FLUSHING AND CLEAN-OUT STRUCTURE

EXISTING

	SURVEY CONTROL STATION
SBDH	STONE BOUND DRILL HOLE
IP	IRON PIPE
X-CUT	CROSS CUT
(R)	RECORD
(M)	MARKED
GP	GRANITE POST
CP	CONCRETE POST
CP	CONCRETE POST
WP	WOOD POST
MPTEL	TELEPHONE MARKER POST
	SIGN
HH	HANDHOLE
	LIGHT POLE
UP	UTILITY POLE
UP/LP	UTILITY POLE/LIGHT POLE
WG	WATER GATE
	HYDRANT
DMH	DRAIN MANHOLE
CB	CATCH BASIN
	TELEPHONE MANHOLE
E	ELECTRIC LINE
D	DRAIN LINE
W	WATER LINE
OHW	OVER-HEAD WIRE
CATV	CABLE TELEVISION
T	TELEPHONE LINE
X 6.8	SPOT ELEVATION
	TREE LINE/HEDGE
	DECIDUOUS TREE
	EVERGREEN TREE
	STONEWALL
	FENCE
	CURBING(TYPE)

ABBREVIATIONS

AC	ASBESTOS CEMENT
CI	CAST IRON
DI	DUCTILE IRON
CU	CONNECTION UNKNOWN
CU	CONNECTION UNKNOWN
ELEC	ELECTRIC
EM	ELECTRIC METER
TEL	TELEPHONE
R	RIM
I	INVERT
CMP	CORRUGATED METAL PIPE
RCP	REINFORCED CONCRETE PIPE
PVC	POLYVINYL CHLORIDE
CPP	CORRUGATED PLASTIC PIPE
TOP	TOP OF PIPE
TOS	TOP OF SILT
TOW	TOP OF WATER
CU	CONNECTION UNKNOWN
MAG	MAGNETIC
FND	FOUND
CLF	CHAIN LINK FENCE
WIF	WROUGHT IRON FENCE
STK	STOCKADE FENCE
PRF	POST AND RAIL FENCE
PKF	PICKET FENCE
LST	LANDSCAPE TIMBER
CSE	COBBLESTONE EDGING
BIT CONC	BITUMINOUS CONCRETE
EOP	EDGE OF PAVEMENT
GRAN	GRANITE
GC	GRANITE CURB
CC	CONCRETE CURB
BCD	BITUMINOUS CONCRETE DRIVE
CD	CONCRETE DRIVE
BCW	BITUMINOUS CONCRETE WALK
GD	GRAVEL DRIVEWAY
CW	CONCRETE WALK
BW	BRICK WALK
FSW	FLAGSTONE WALK
BRW	BRICK RETAINING WALL
TRW	TIMBER RETAINING WALL
CRW	CONCRETE RETAINING WALL
SRW	STONE RETAINING WALL
CCPAD	CONCRETE PAD
PL	PLANTED

REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: J. HERMAN
DRAWN BY: J. HERMAN
SHEET CHK'D BY: M. GUIDICE
CROSS CHK'D BY: C. KERSHAW
APPROVED BY: M. GUIDICE
DATE: JULY 2017

CDM Smith
260 West Exchange Street, Suite 300
Providence, RI 02903
Tel: (401) 751-5360

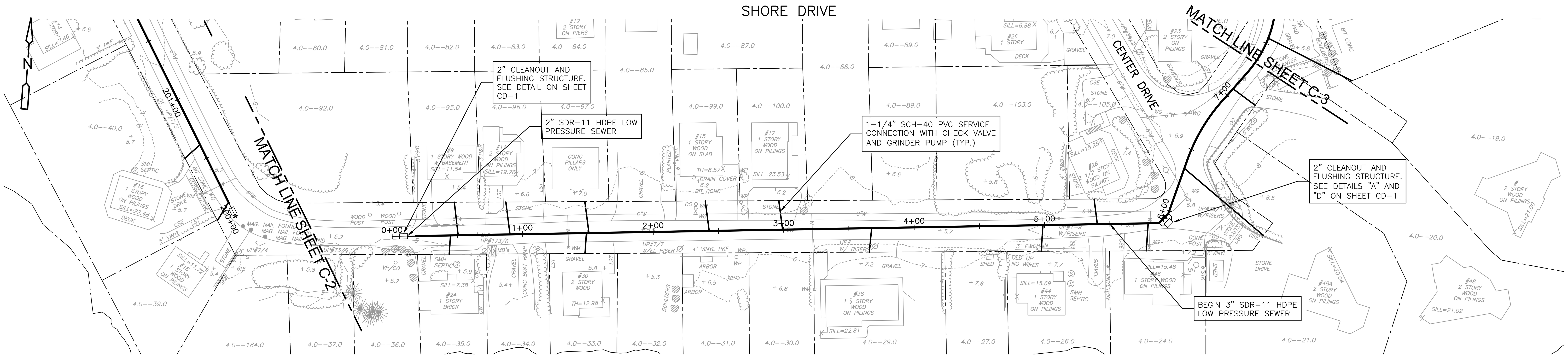
TOWN OF MARION, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

GENERAL NOTES AND LEGEND

PROJECT NO. 0357-114884
FILE NAME: G001GELG.DWG
SHEET NO. G-1

30% SUBMITTAL - NOT FOR CONSTRUCTION



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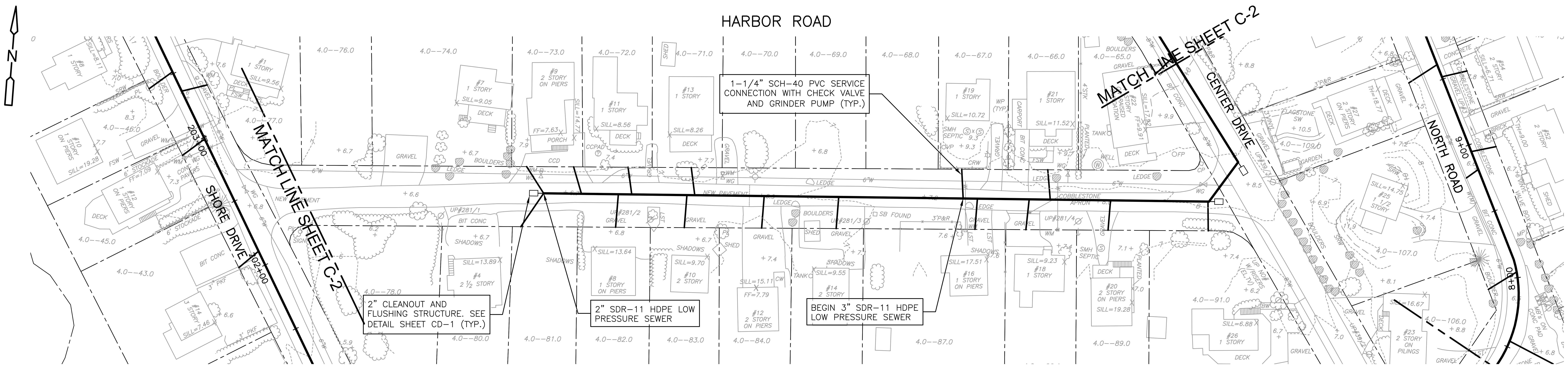

PLAN
1" = 40'

REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: J. HERMAN
DRAWN BY: L. KERSHAW
SHEET CHK'D BY: M. GUIDICE
CROSS CHK'D BY: C. KERSHAW
APPROVED BY: M. GUIDICE
DATE: JULY 2017

CDM Smith
260 West Exchange Street, Suite 300
Providence, RI 02903
Tel: (401) 751-5360

TOWN OF MATTAPOISETT, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

SHORE DRIVE AND HARBOR ROAD

PROJECT NO. 0357-114884
FILE NAME: C001STPL.DWG
SHEET NO.
C-1

30% SUBMITTAL - NOT FOR CONSTRUCTION

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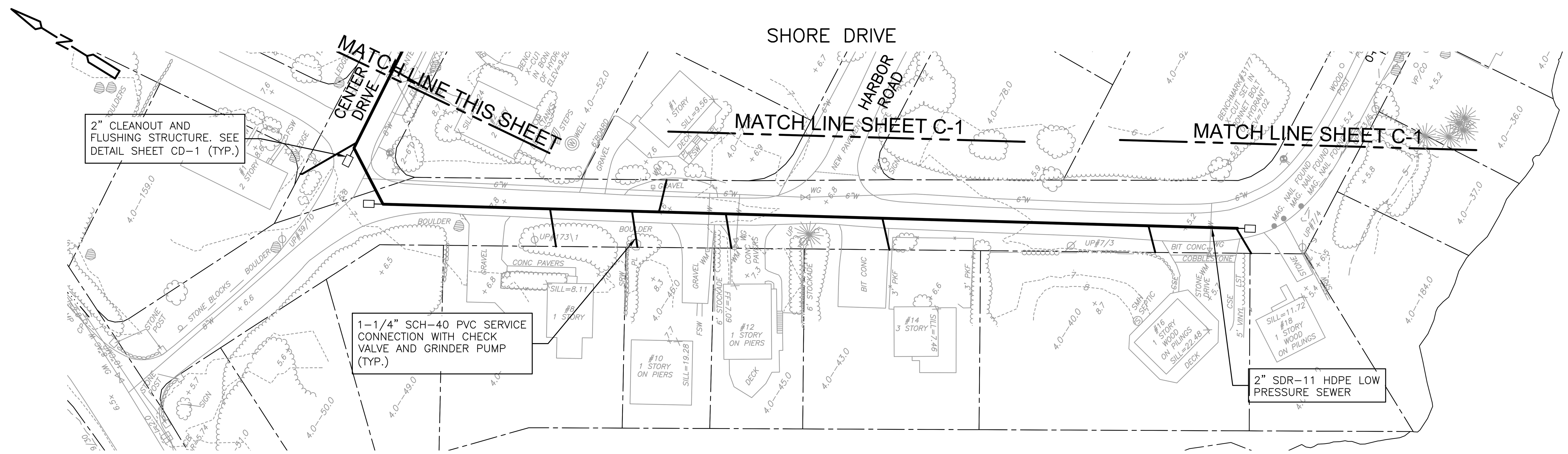

PLAN
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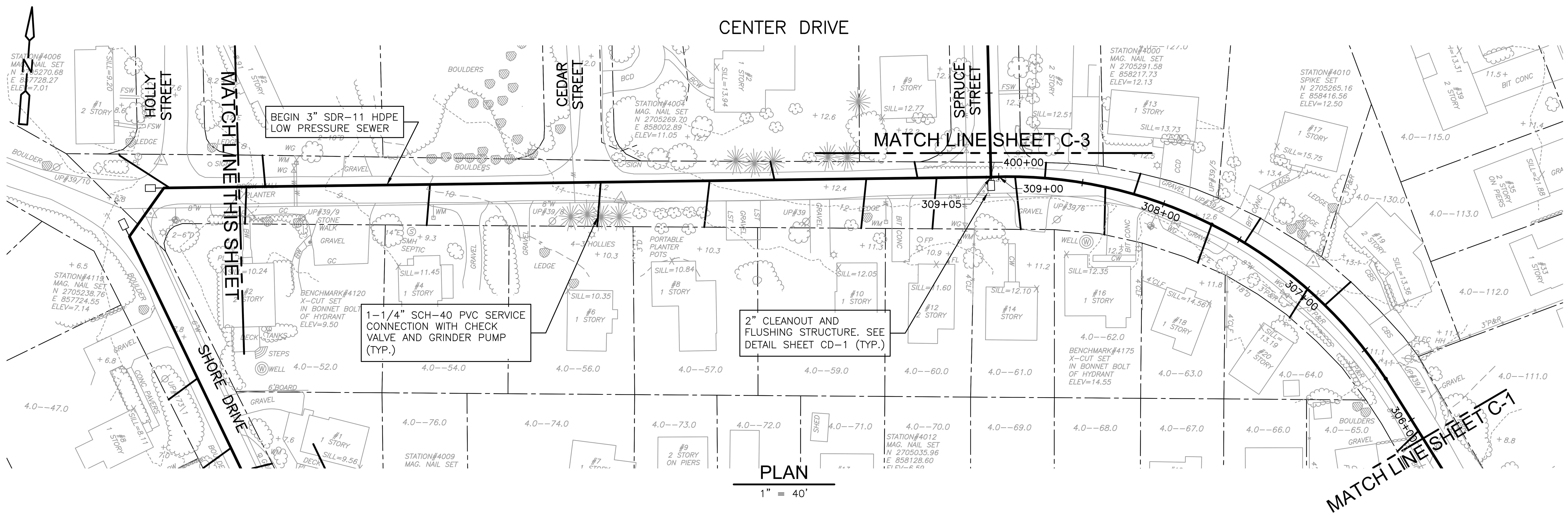


PLAN
1" = 40'

REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY:	J. HERMAN
DRAWN BY:	L. KERSHAW
SHEET CHK'D BY:	M. GUIDICE
CROSS CHK'D BY:	C. KERSHAW
APPROVED BY:	M. GUIDICE
DATE:	JULY 2017

CDM Smith
260 West Exchange Street, Suite 300
Providence, RI 02903
Tel: (401) 751-5360

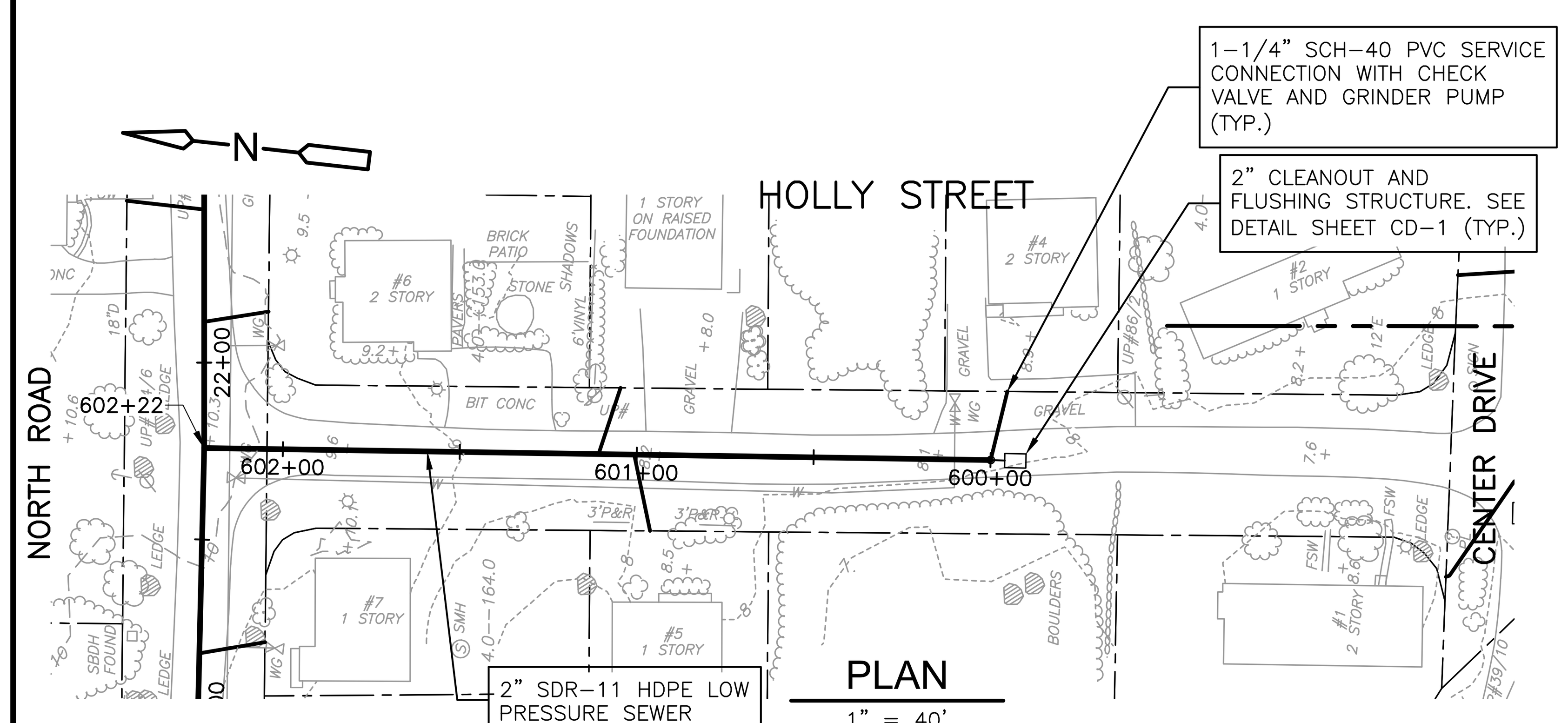
TOWN OF MATTAPOISETT, MASSACHUSETTS

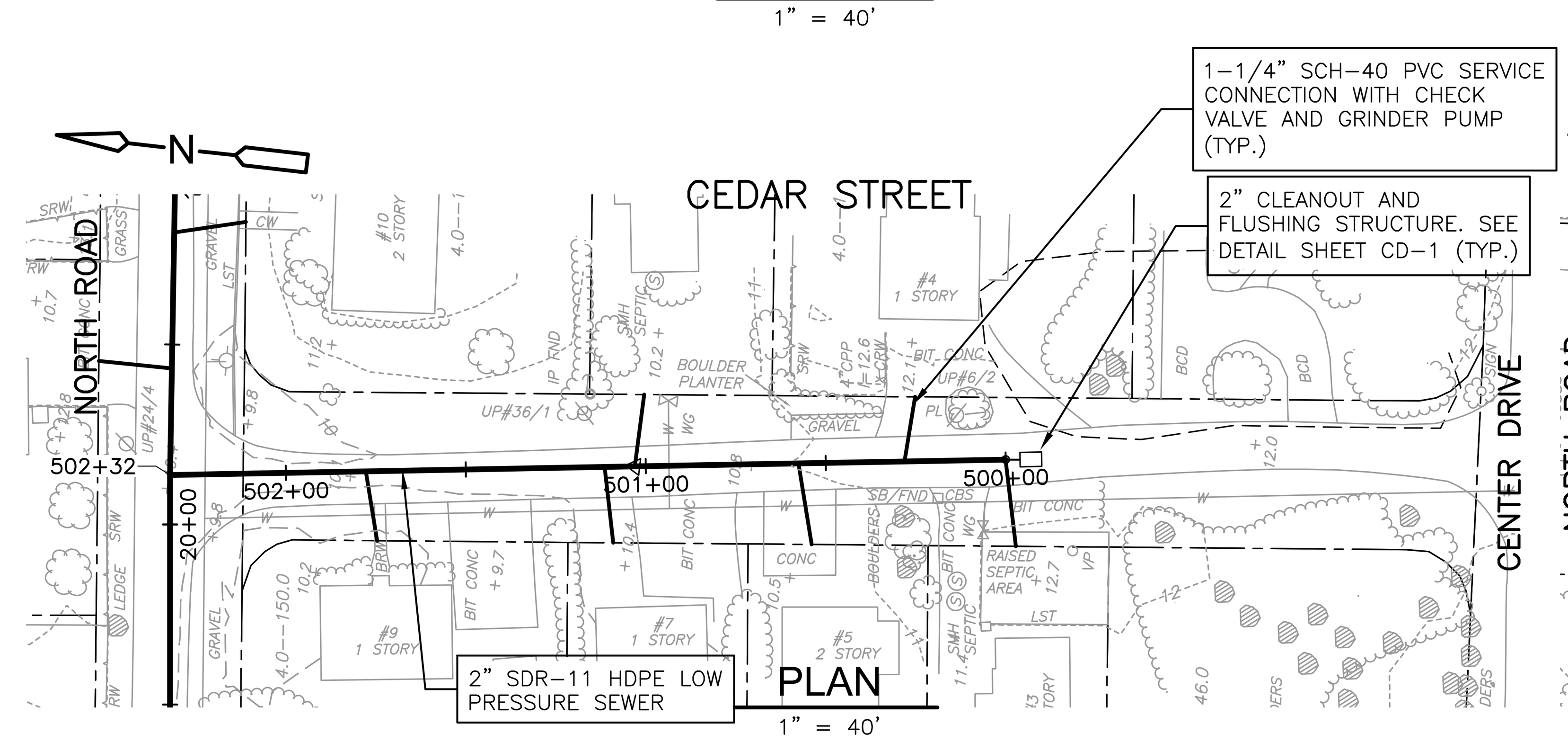
AUCOOT COVE SEWER EVALUATION

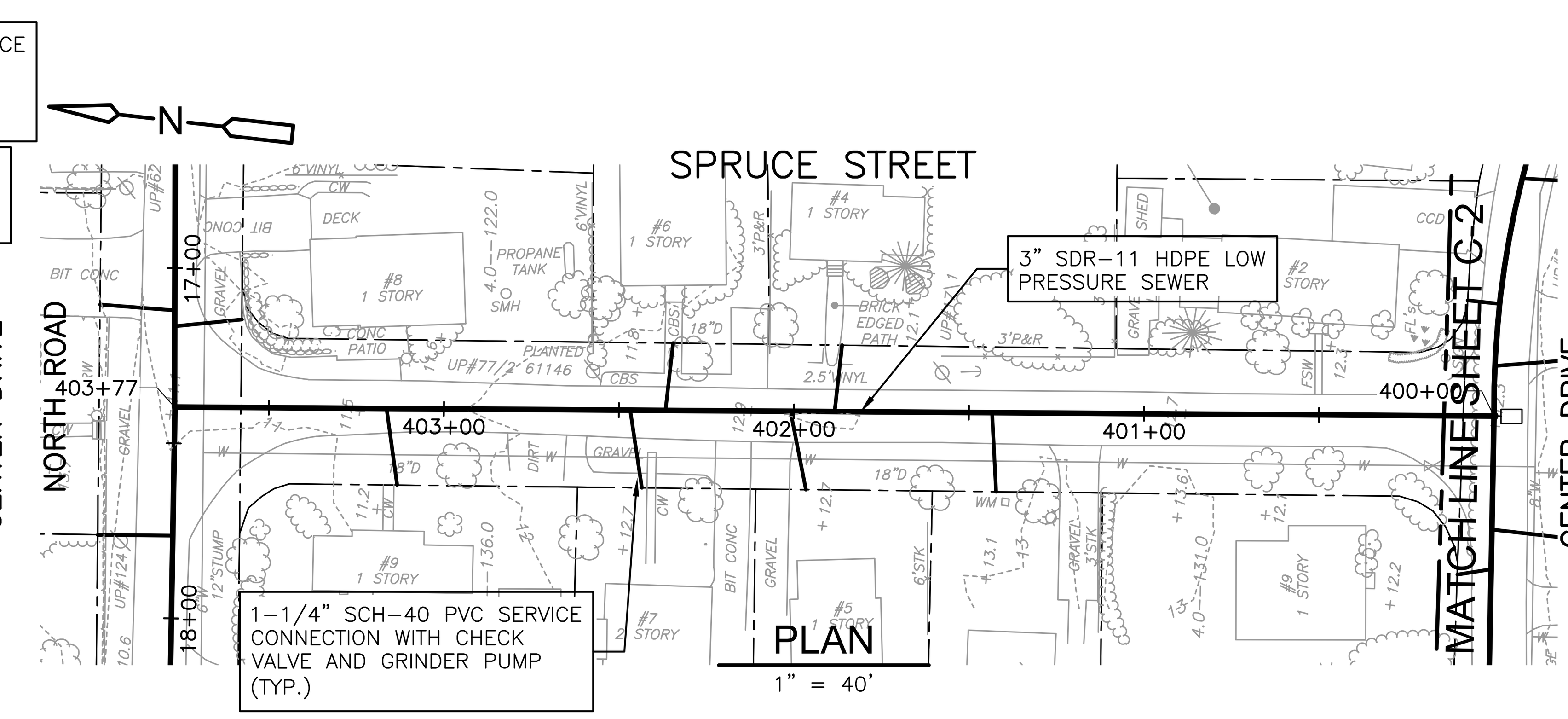
SHORE DRIVE AND CENTER DRIVE

PROJECT NO. 0357-114884
FILE NAME: C002STPL.DWG
SHEET NO.
C-2

30% SUBMITTAL - NOT FOR CONSTRUCTION







REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: J. HERMAN
DRAWN BY: L. KERSHAW
SHEET CHK'D BY: M. GUIDICE
CROSS CHK'D BY: C. KERSHAW
APPROVED BY: M. GUIDICE
DATE: JULY 2017

CDM Smith
260 West Exchange Street, Suite 300
Providence, RI 02903
Tel: (401) 751-5360

TOWN OF MATTAPOISETT, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

NORTH ROAD, HOLLY STREET, CEDAR STREET, AND SPRUCE STREET

PROJECT NO. 0357-114884
FILE NAME: C003STPL.DWG
SHEET NO. C-3

30% SUBMITTAL - NOT FOR CONSTRUCTION

XREFs: [CDMS_2436, 20161100IWS3, CSTBP001] Images: []
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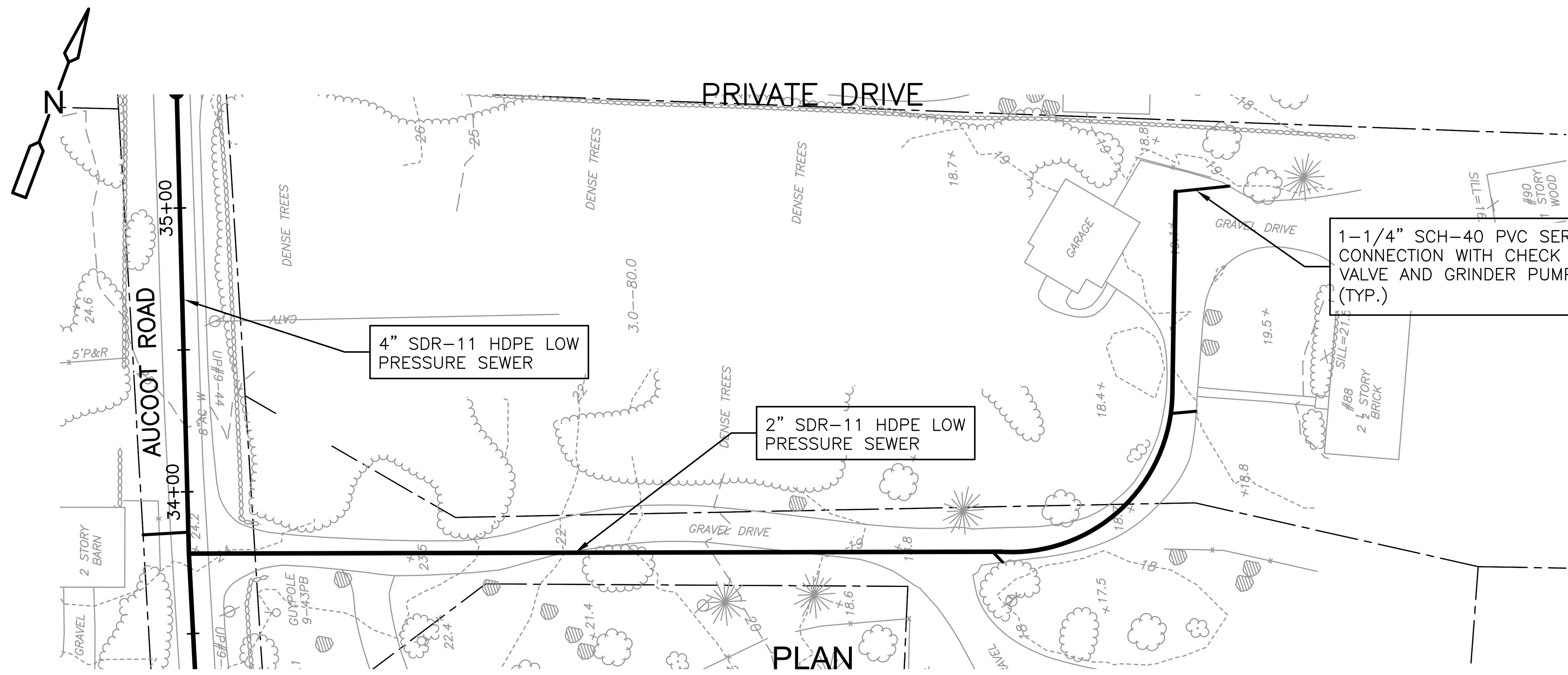

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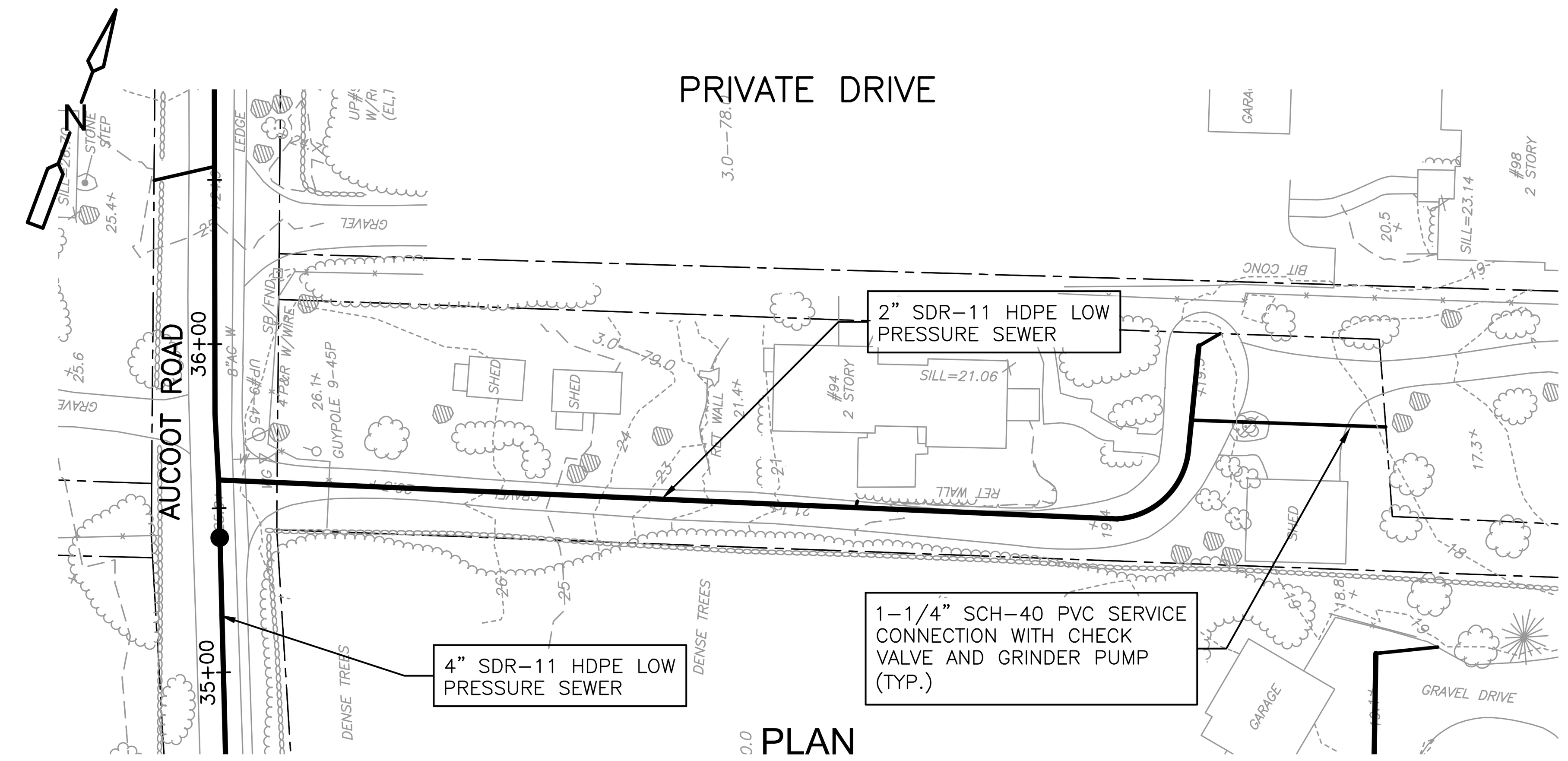

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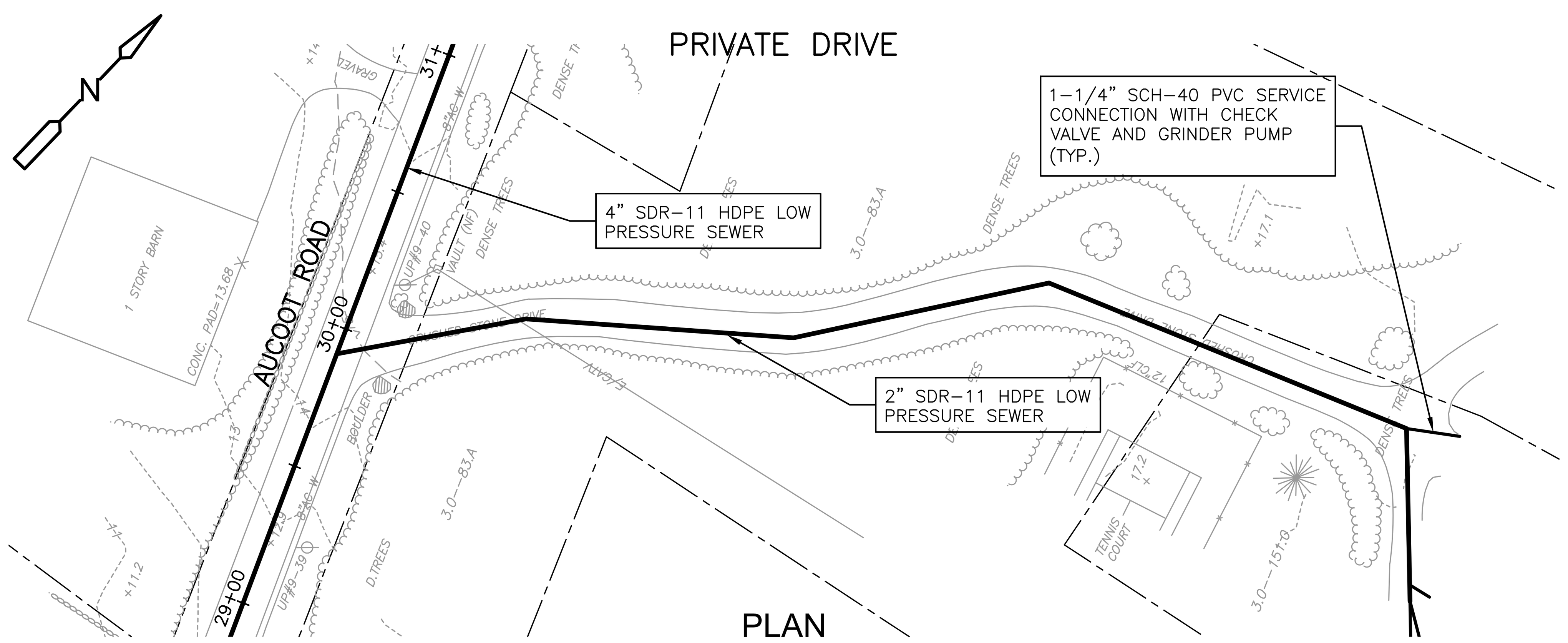

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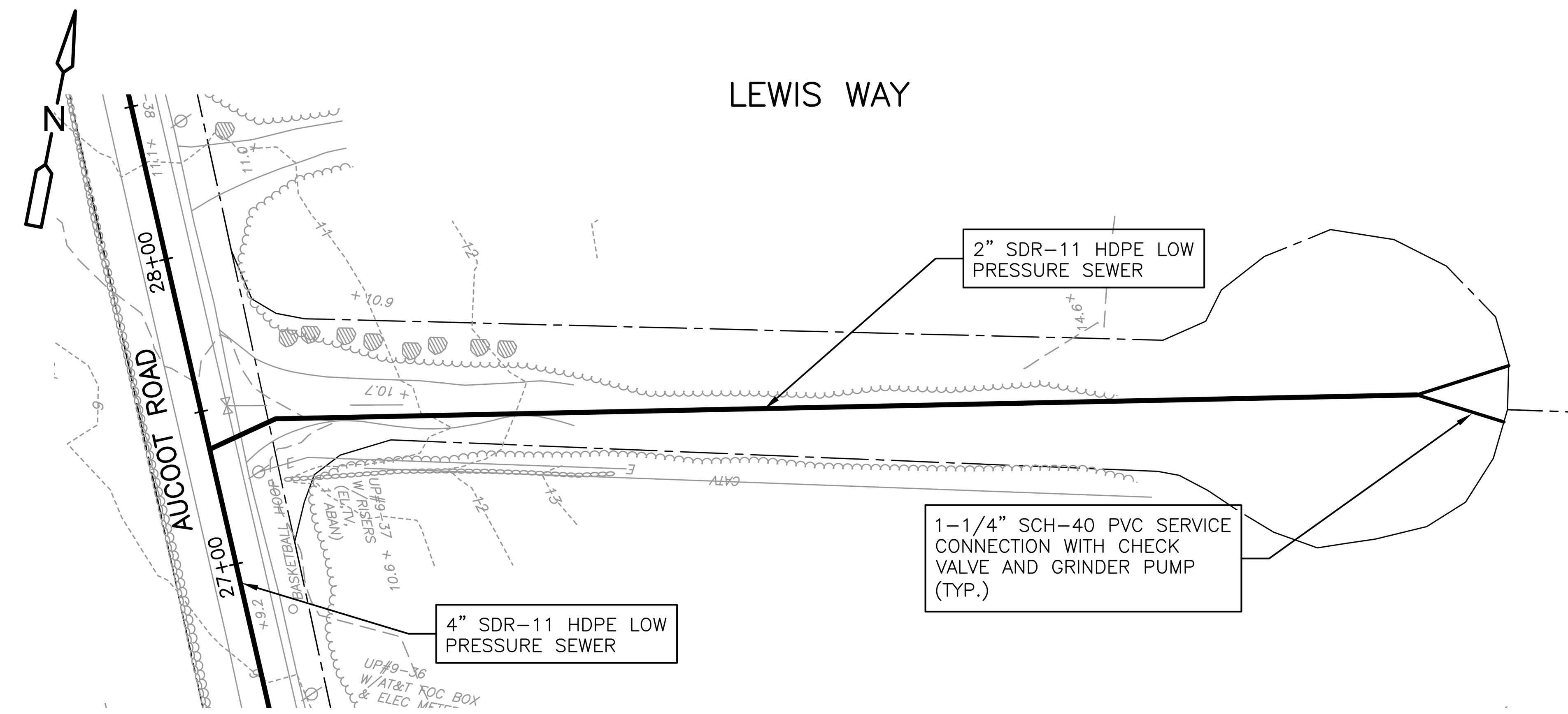

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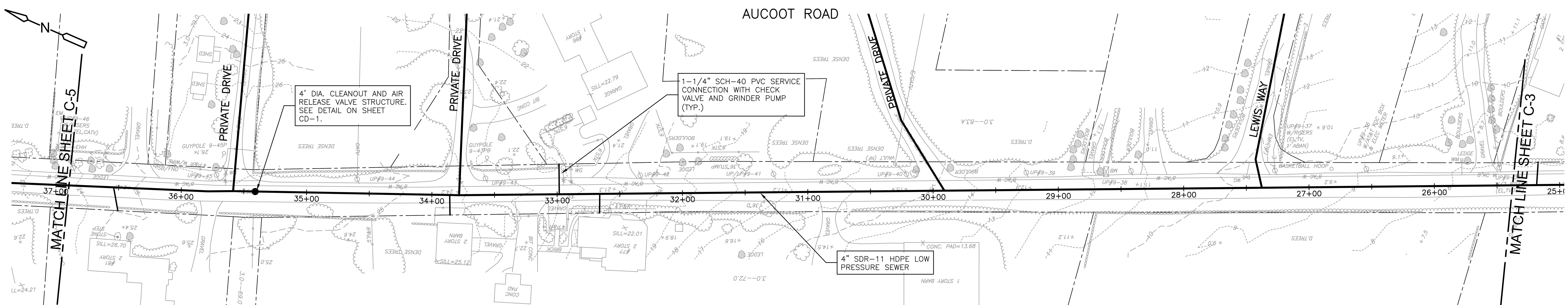


PLAN
1" = 40'

REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: J. HERMAN
DRAWN BY: L. KERSHAW
SHEET CHK'D BY: M. GUIDICE
CROSS CHK'D BY: C. KERSHAW
APPROVED BY: M. GUIDICE
DATE: JULY 2017

CDM Smith
260 West Exchange Street, Suite 300
Providence, RI 02903
Tel: (401) 751-5360

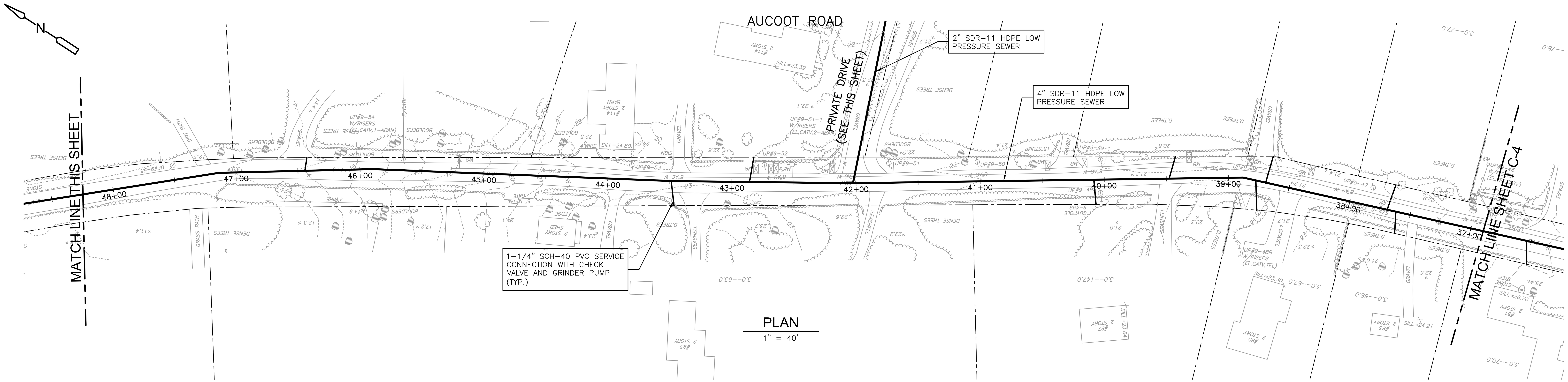
TOWN OF MATTAPOISETT, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

AUCOOT ROAD I

PROJECT NO. 0357-114884
FILE NAME: C004STPL.DWG
SHEET NO.
C-4

30% SUBMITTAL - NOT FOR CONSTRUCTION



PLAN
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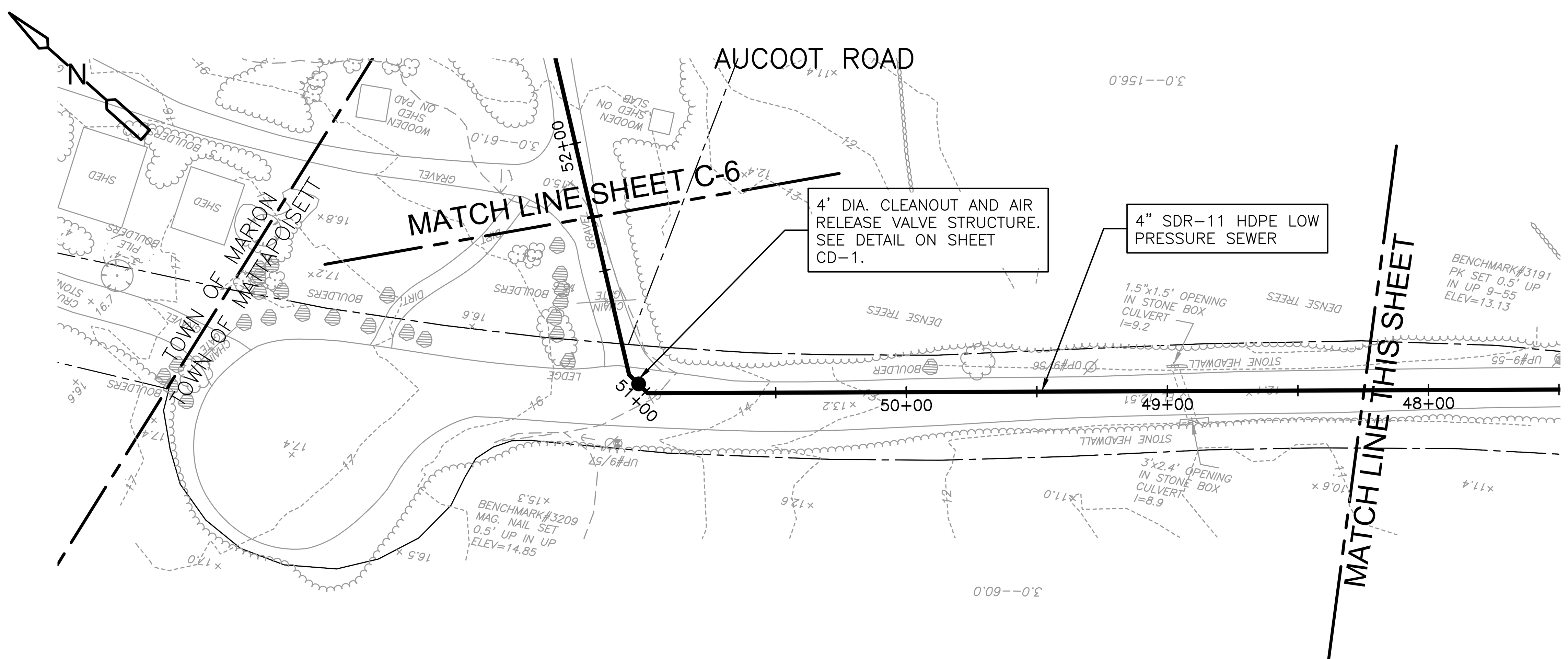
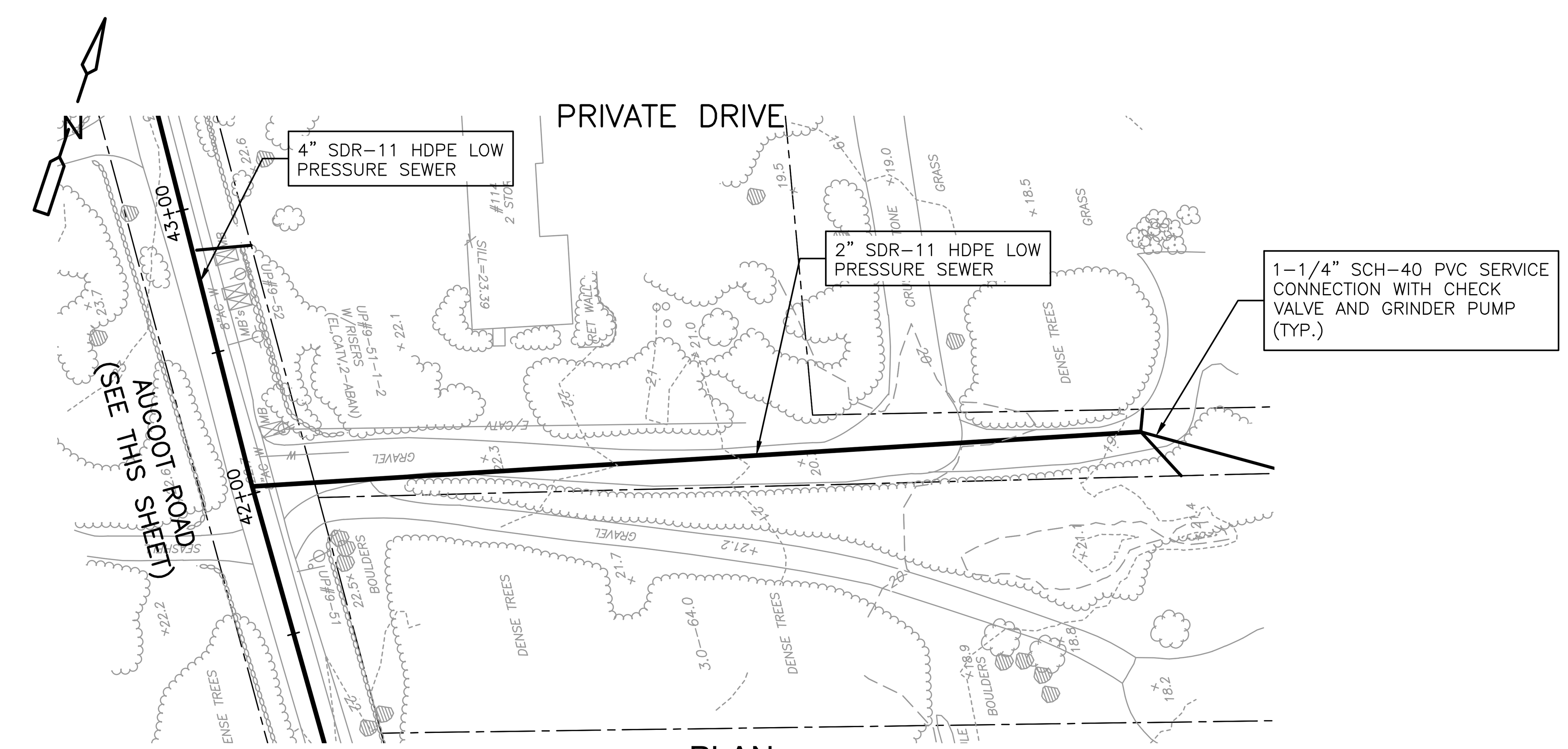


PLAN
1" = 40'



PLAN
1" = 40'

REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: J. HERMAN
DRAWN BY: L. KERSHAW
SHEET CHK'D BY: M. GUIDICE
CROSS CHK'D BY: C. KERSHAW
APPROVED BY: M. GUIDICE
DATE: JULY 2017

CDM Smith
260 West Exchange Street, Suite 300
Providence, RI 02903
Tel: (401) 751-5360

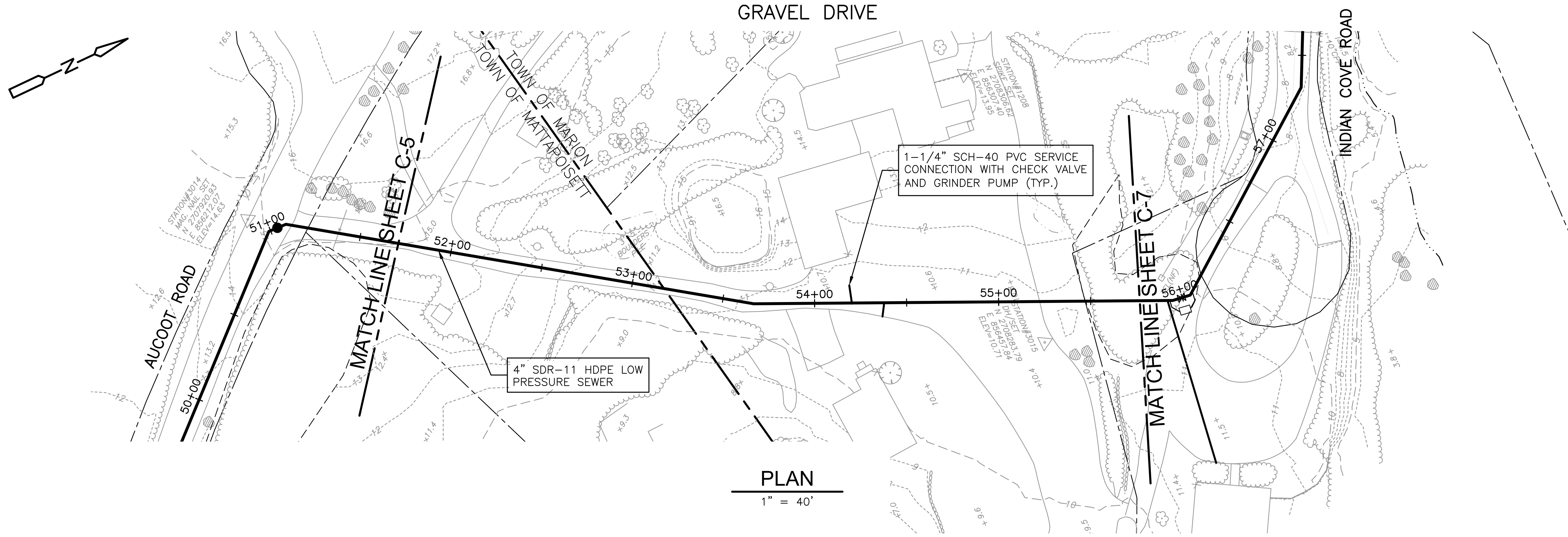
TOWN OF MATTAPOISETT, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

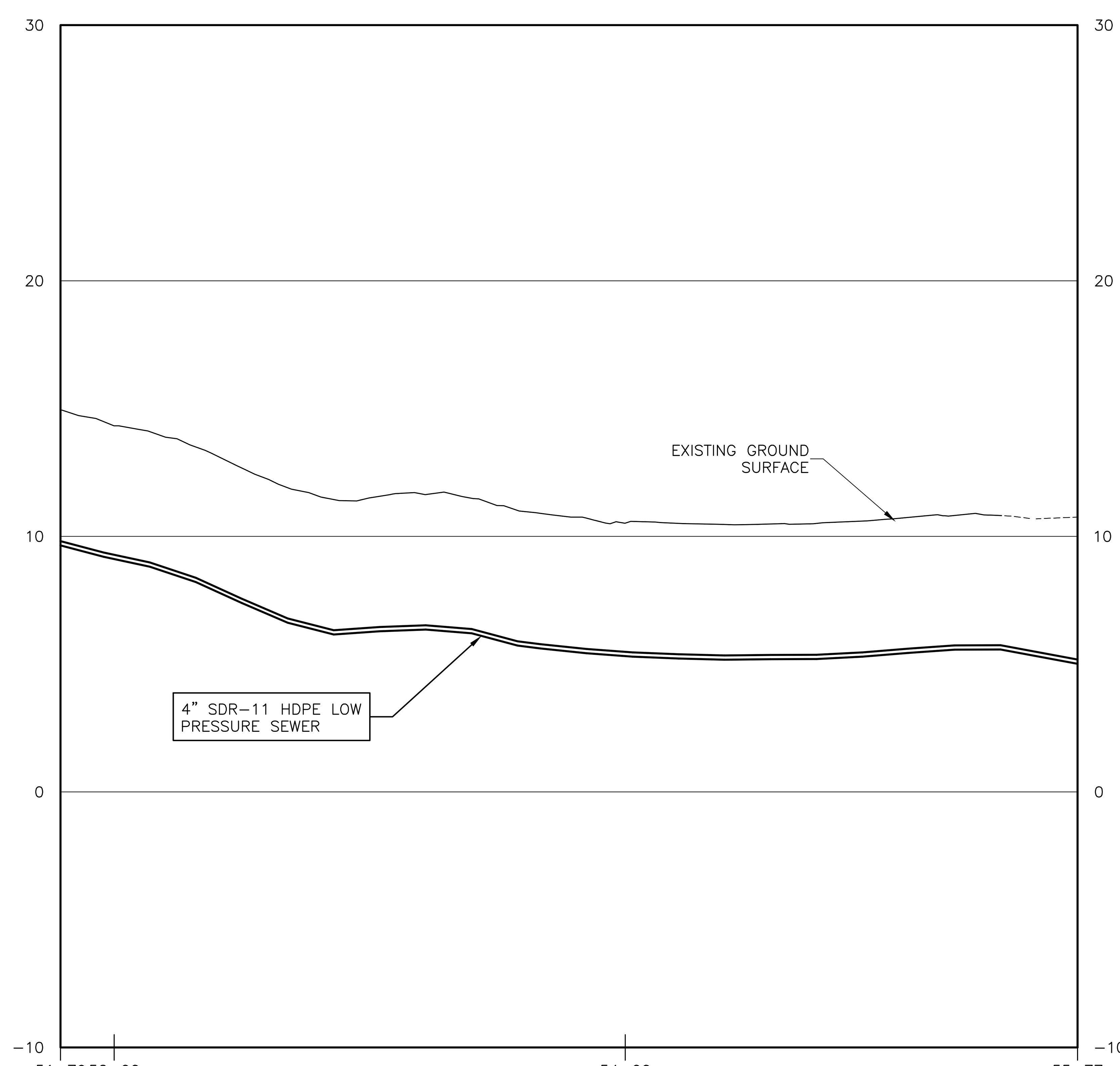
AUCOOT ROAD II

PROJECT NO. 0357-114884
FILE NAME: C005STPL.DWG
SHEET NO.
C-5

30% SUBMITTAL - NOT FOR CONSTRUCTION



PLAN
1" = 40'



1" = 40'
20 0 40

REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: J. HERMAN
DRAWN BY: L. KERSHAW
SHEET CHK'D BY: M. GUIDICE
CROSS CHK'D BY: C. KERSHAW
APPROVED BY: M. GUIDICE
DATE: JULY 2017

CDM Smith
260 West Exchange Street, Suite 300
Providence, RI 02903
Tel: (401) 751-5360

TOWN OF MARION, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

AUCOOT ROAD CROSS COUNTRY TO INDIAN COVE ROAD

PROJECT NO. 0357-114884
FILE NAME: C006STPL.DWG
SHEET NO.
C-6

50% SUBMITTAL - NOT FOR CONSTRUCTION

XREFs: [CDMS_2436, 20161100IWS3, CSTBP001] Images: []
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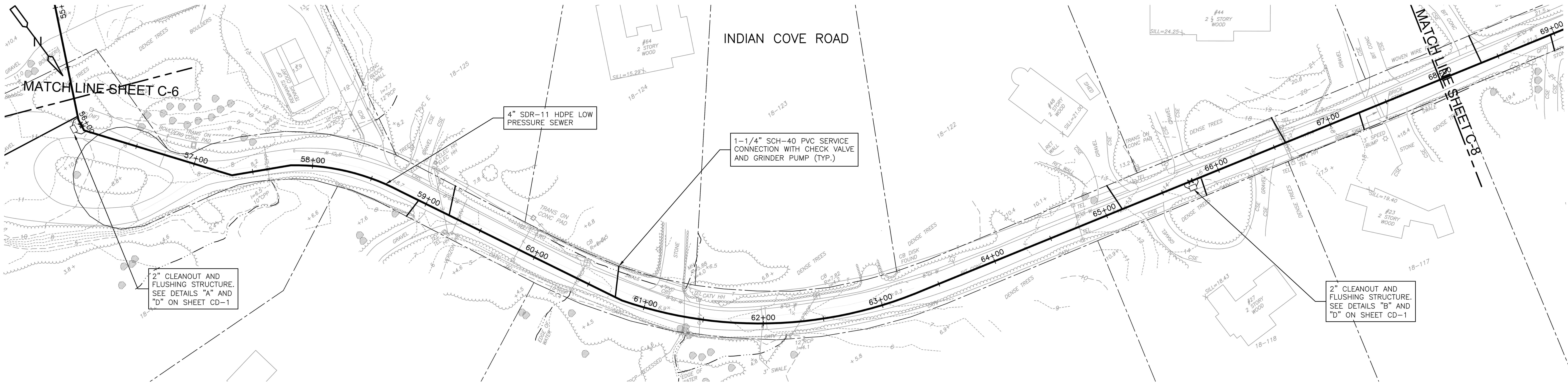
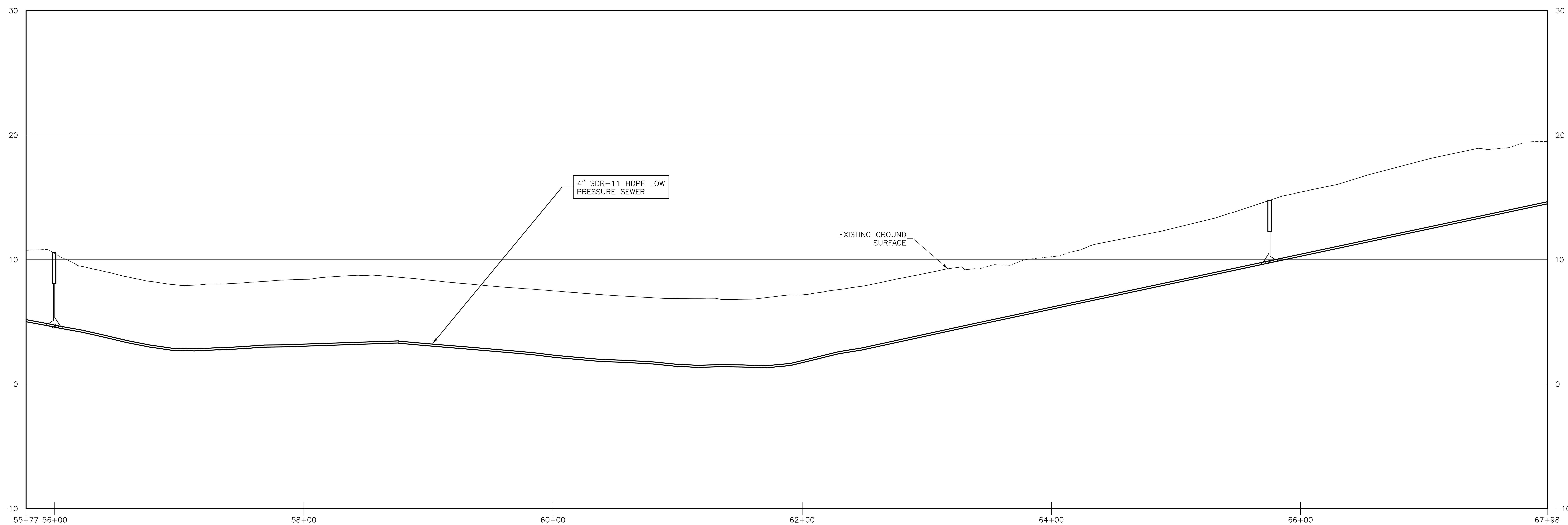


PLAN
1" = 40'



1" = 40'

REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: J. HERMAN
DRAWN BY: L. KERSHAW
SHEET CHK'D BY: M. GUIDICE
CROSS CHK'D BY: C. KERSHAW
APPROVED BY: M. GUIDICE
DATE: JULY 2017

CDM Smith
260 West Exchange Street, Suite 300
Providence, RI 02903
Tel: (401) 751-5360

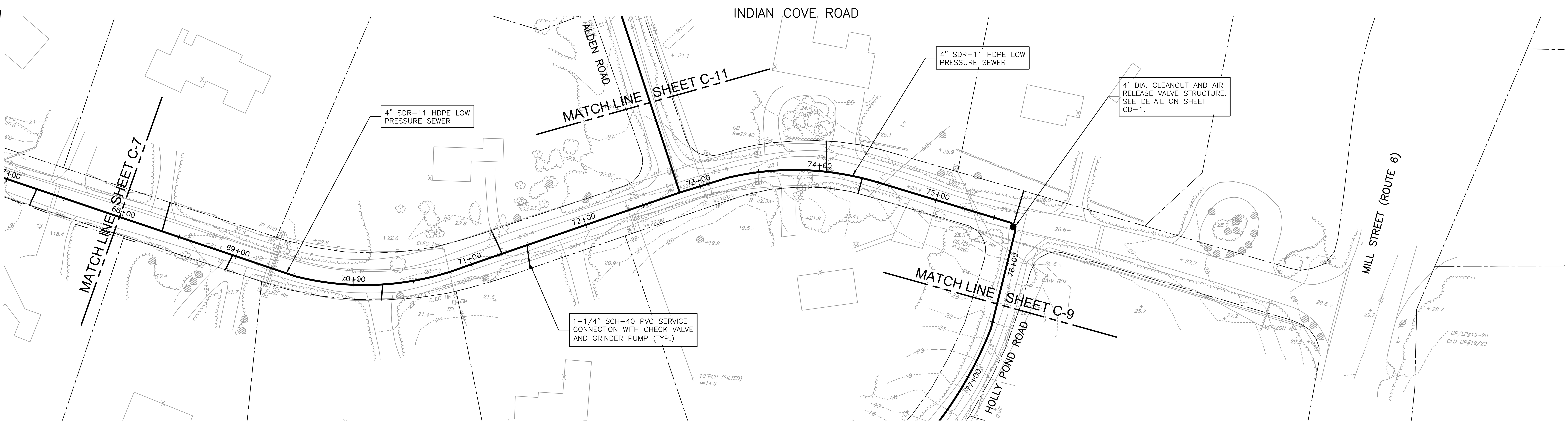
TOWN OF MARION, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

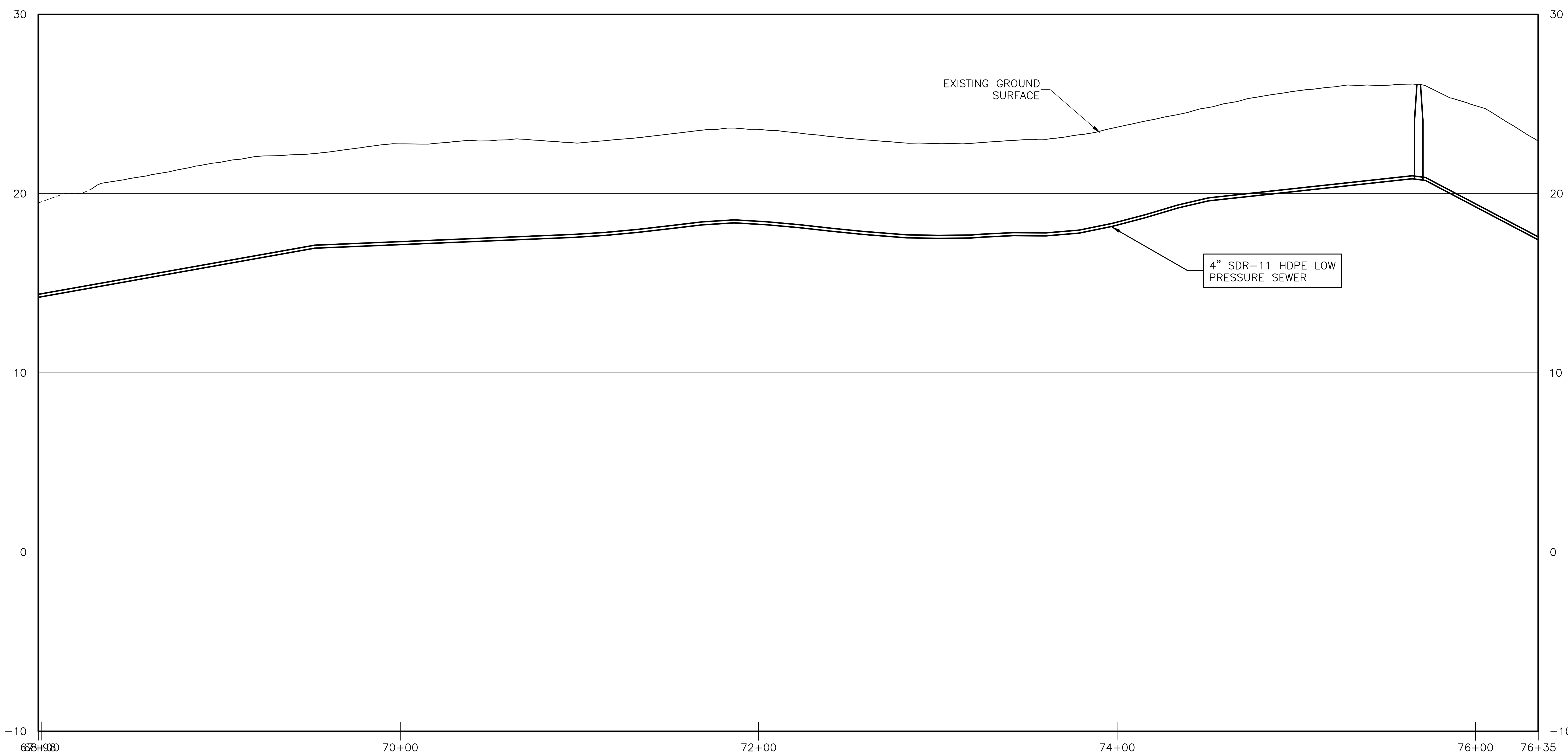
INDIAN COVE ROAD I

PROJECT NO. 0357-114884
FILE NAME: C007STPL.DWG
SHEET NO.
C-7

50% SUBMITTAL - NOT FOR CONSTRUCTION



PLAN
1" = 40'



REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: J. HERMAN
DRAWN BY: L. KERSHAW
SHEET CHK'D BY: M. GUIDICE
CROSS CHK'D BY: C. KERSHAW
APPROVED BY: M. GUIDICE
DATE: JULY 2017

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260 West Exchange Street, Suite 300
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TOWN OF MARION, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

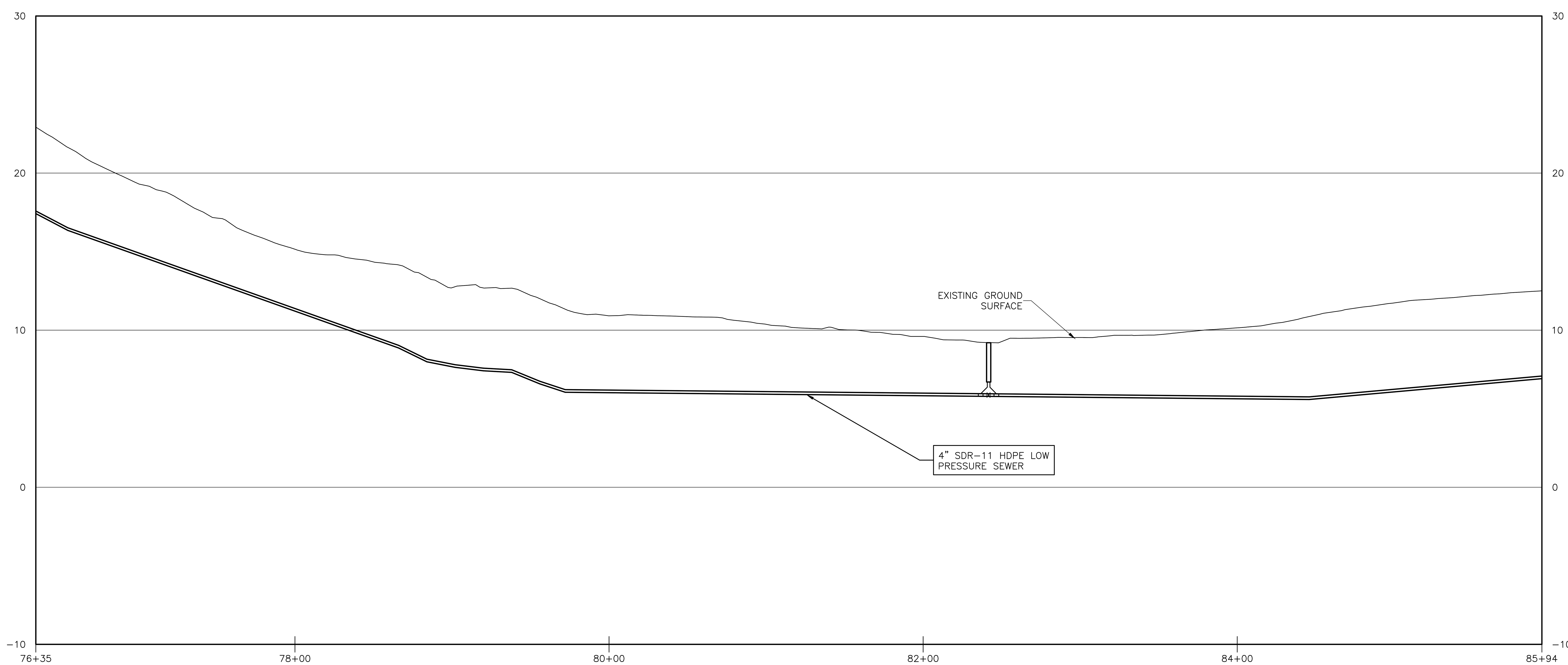
INDIAN COVE ROAD II

PROJECT NO. 0357-114884
FILE NAME: C008STPL.DWG
SHEET NO.
C-8

50% SUBMITTAL - NOT FOR CONSTRUCTION

PLAN

1" = 40'



REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: J. HERMAN
DRAWN BY: L. KERSHAW
SHEET CHK'D BY: M. GUIDICE
CROSS CHK'D BY: C. KERSHAW
APPROVED BY: M. GUIDICE
DATE: JULY 2017

CDM Smith
260 West Exchange Street, Suite 300
Providence, RI 02903
Tel: (401) 751-5360

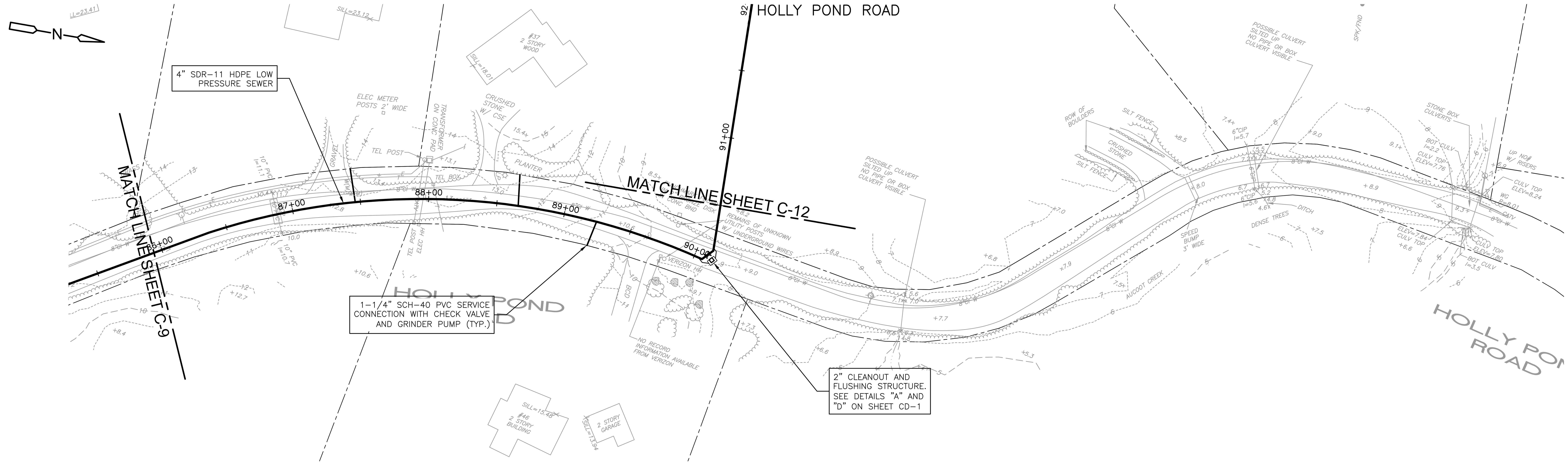
TOWN OF MARION, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

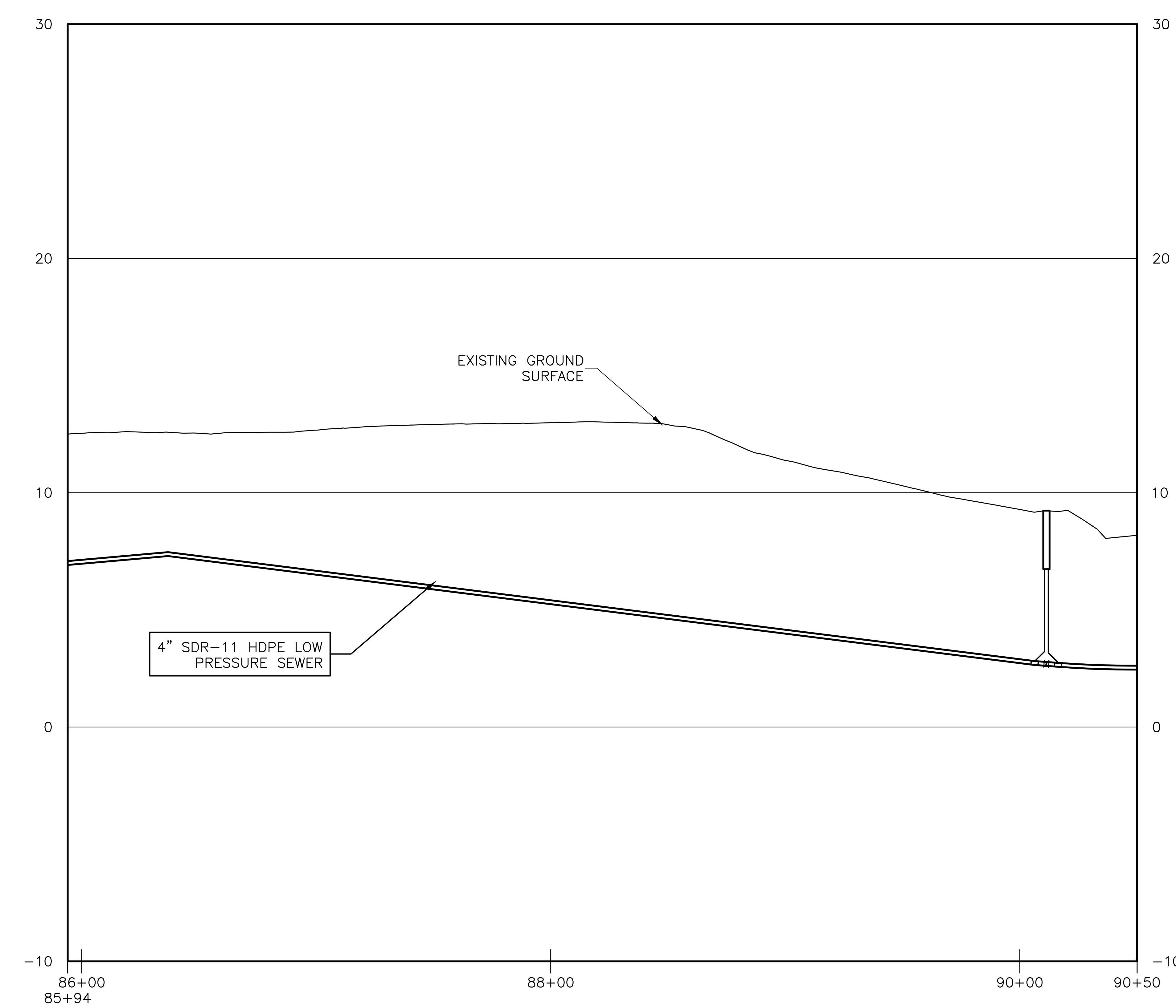
HOLLY POND ROAD I

PROJECT NO. 0357-114884
FILE NAME: C009STPL.DWG
SHEET NO.
C-9

50% SUBMITTAL - NOT FOR CONSTRUCTION



PLAN
1" = 40'



REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: J. HERMAN
DRAWN BY: L. KERSHAW
SHEET CHK'D BY: M. GUIDICE
CROSS CHK'D BY: C. KERSHAW
APPROVED BY: M. GUIDICE
DATE: JULY 2017

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TOWN OF MARION, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

HOLLY POND ROAD II

PROJECT NO. 0357-114884
FILE NAME: C010STPL.DWG
SHEET NO.
C-10

50% SUBMITTAL - NOT FOR CONSTRUCTION

XREFs: [CDMS_2436, 20161101WS3, CSTBP001] Images: []
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REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: J. HERMAN
DRAWN BY: L. KERSHAW
SHEET CHK'D BY: M. GUIDICE
CROSS CHK'D BY: C. KERSHAW
APPROVED BY: M. GUIDICE
DATE: JULY 2017

CDM Smith
260 West Exchange Street, Suite 300
Providence, RI 02903
Tel: (401) 751-5360

TOWN OF MARION, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

SASSAMON TRAIL AND ALDEN ROAD

PROJECT NO. 0357-114884
FILE NAME: C011STPL.DWG
SHEET NO.
C-11

50% SUBMITTAL - NOT FOR CONSTRUCTION

XREFs: [CDMS_2436, 20161100IWS3, CSTBP001] Images: []
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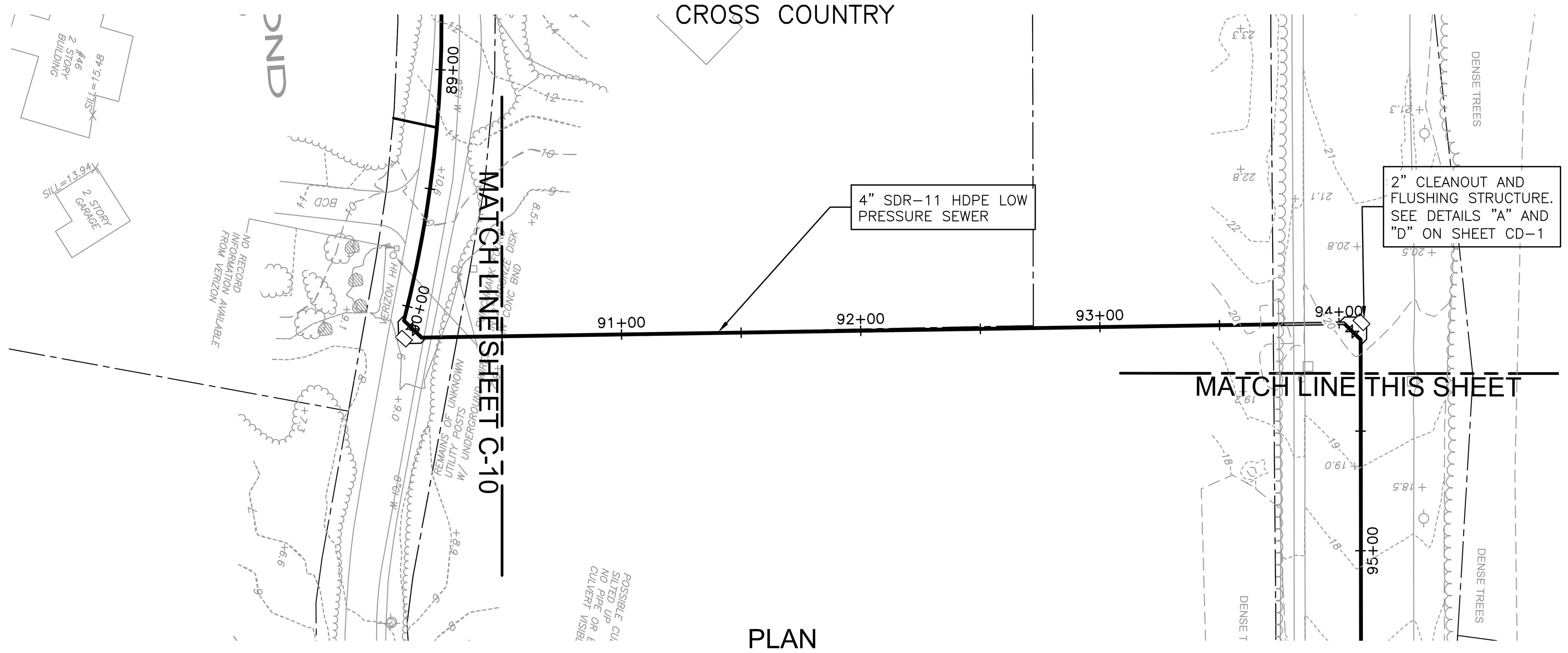
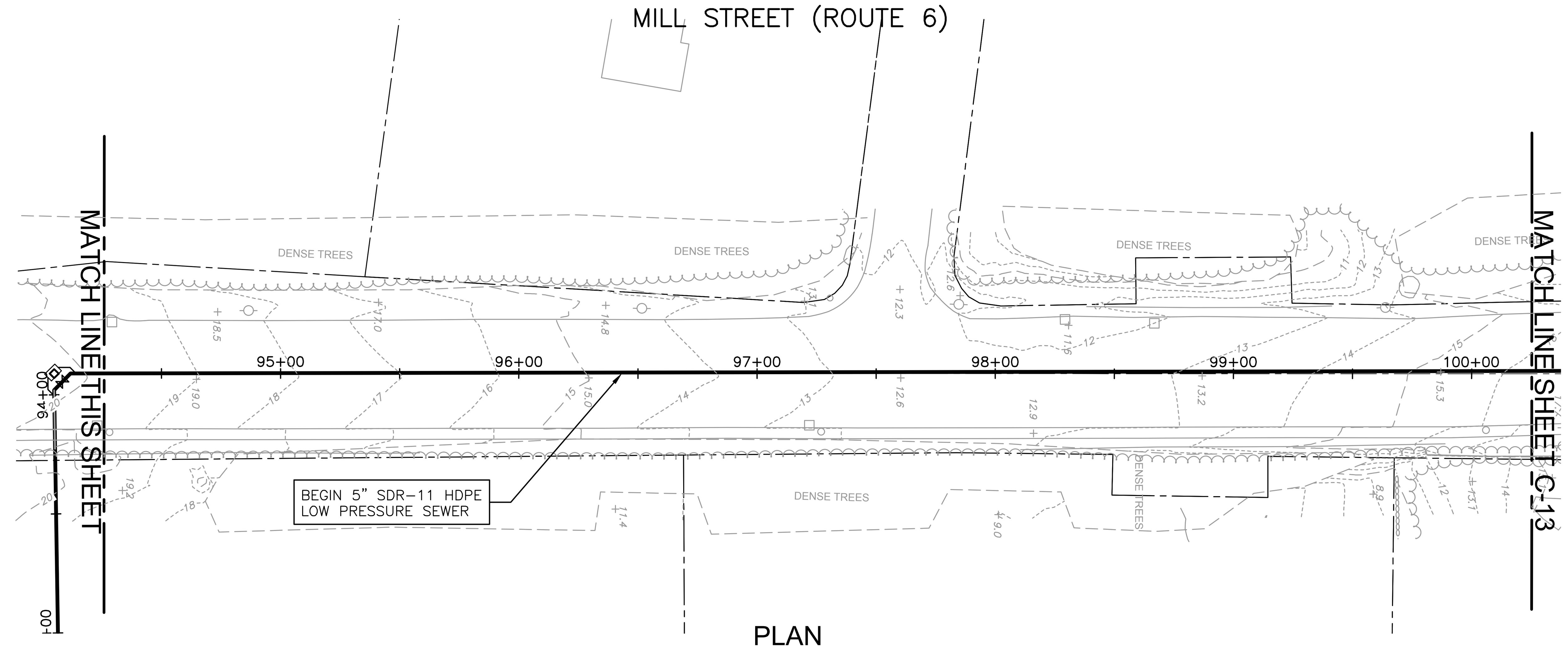
CROSS COUNTRY



PLAN
1" = 40'

MILL STREET (ROUTE 6)



PLAN
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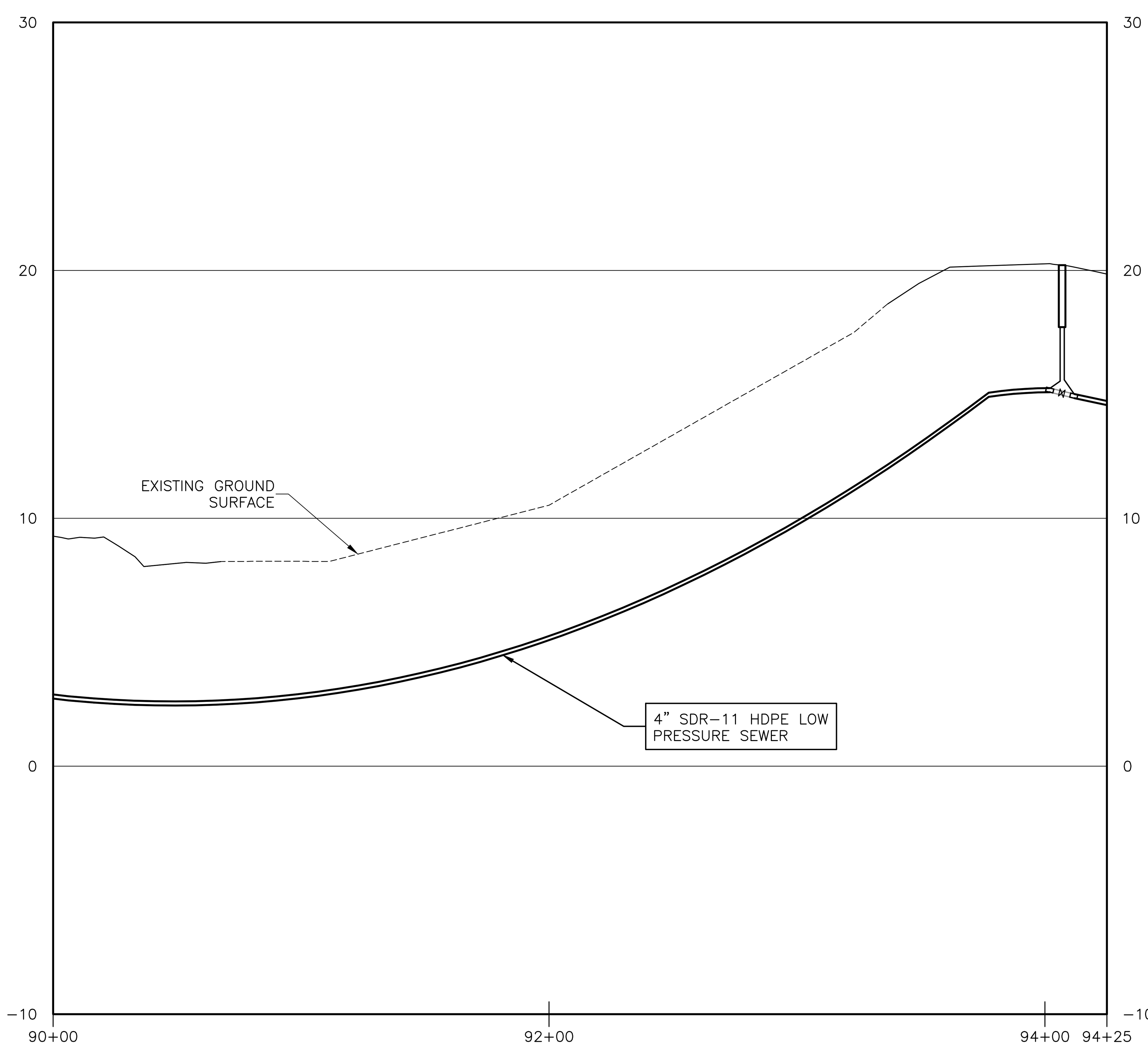

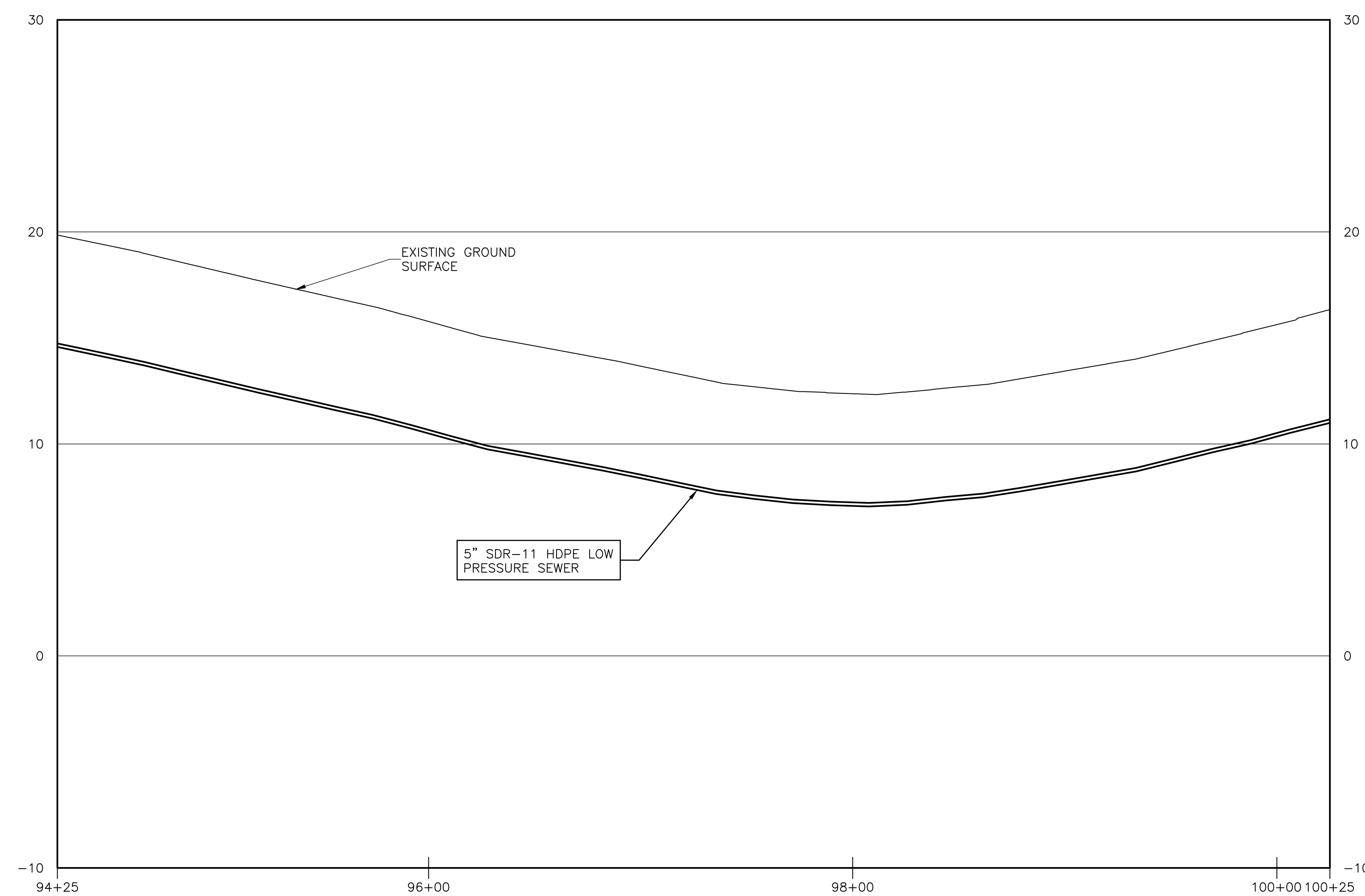

REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: J. HERMAN
DRAWN BY: L. KERSHAW
SHEET CHK'D BY: M. GUIDICE
CROSS CHK'D BY: C. KERSHAW
APPROVED BY: M. GUIDICE
DATE: JULY 2017

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TOWN OF MARION, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

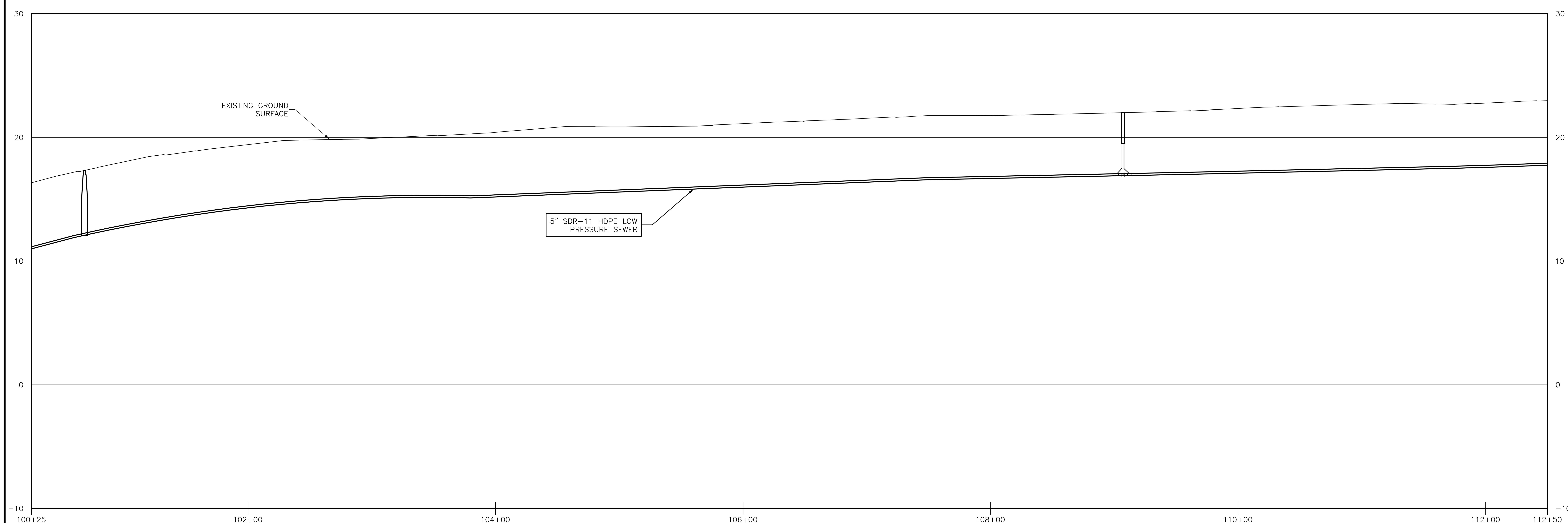
MILL STREET (ROUTE 6) I

PROJECT NO. 0357-114884
FILE NAME: C012STPL.DWG
SHEET NO.
C-12

50% SUBMITTAL - NOT FOR CONSTRUCTION

MILL STREET (ROUTE 6)

PLAN
1" = 40'



REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: J. HERMAN
DRAWN BY: L. KERSHAW
SHEET CHK'D BY: M. GUIDICE
CROSS CHK'D BY: C. KERSHAW
APPROVED BY: M. GUIDICE
DATE: JULY 2017

CDM Smith
260 West Exchange Street, Suite 300
Providence, RI 02903
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TOWN OF MARION, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

MILL STREET (ROUTE 6) II

PROJECT NO. 0357-114884
FILE NAME: C013STPL.DWG
SHEET NO.
C-13

50% SUBMITTAL - NOT FOR CONSTRUCTION

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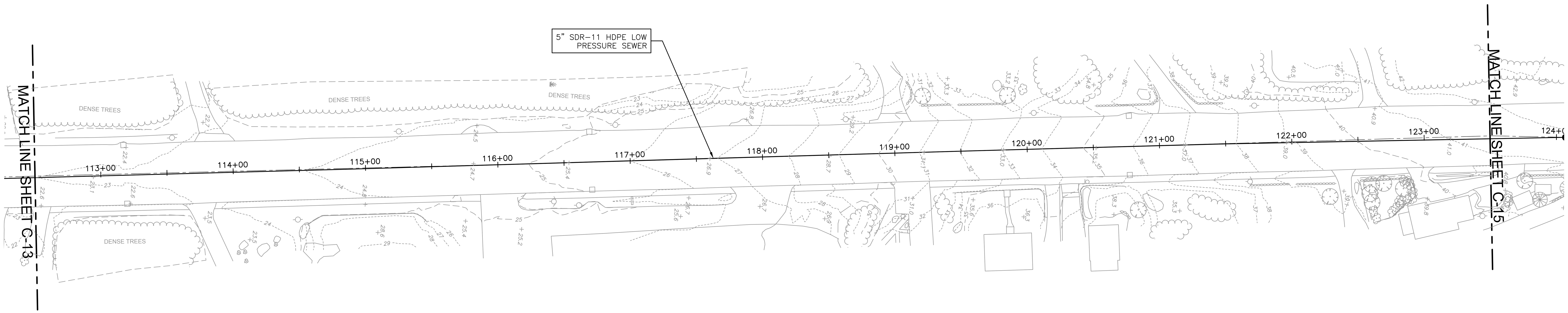
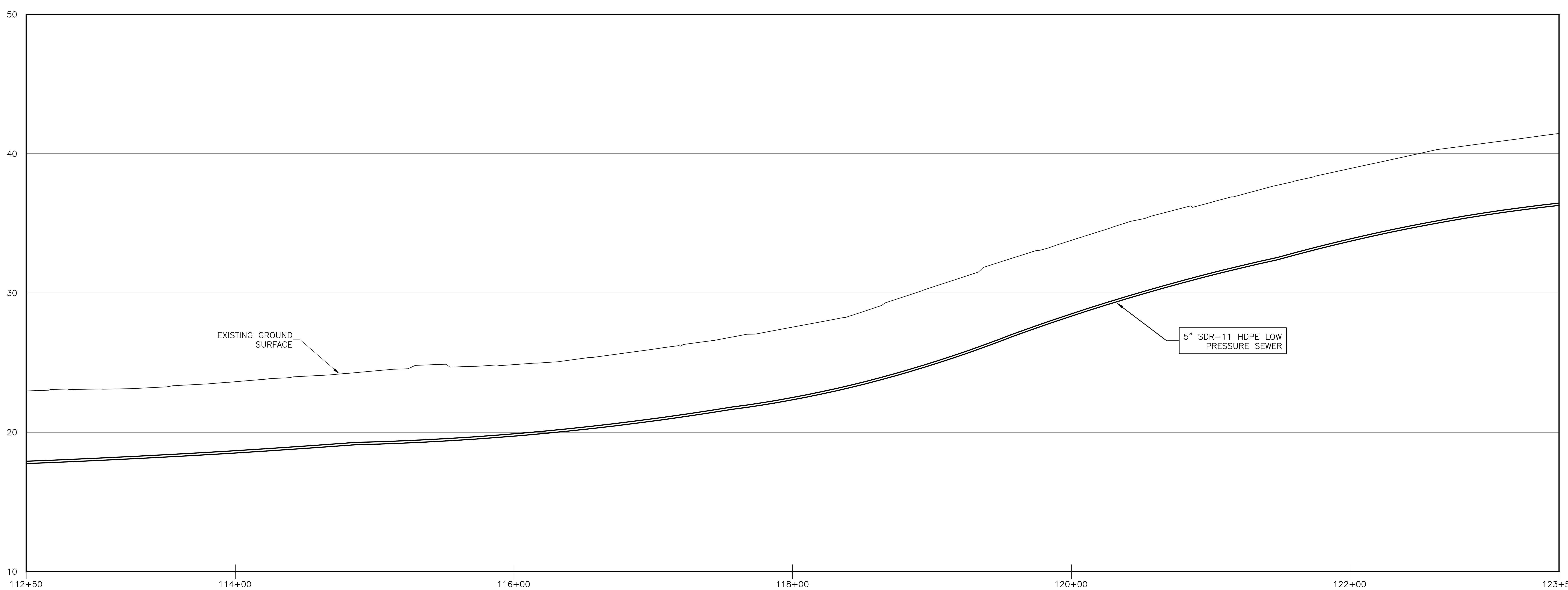
MILL STREET (ROUTE 6)



PLAN
1" = 40'



REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: J. HERMAN
DRAWN BY: L. KERSHAW
SHEET CHK'D BY: M. GUIDICE
CROSS CHK'D BY: C. KERSHAW
APPROVED BY: M. GUIDICE
DATE: JULY 2017

CDM Smith
260 West Exchange Street, Suite 300
Providence, RI 02903
Tel: (401) 751-5360

TOWN OF MARION, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

MILL STREET (ROUTE 6) II

PROJECT NO. 0357-114884
FILE NAME: C014STPL.DWG
SHEET NO.
C-14

50% SUBMITTAL - NOT FOR CONSTRUCTION

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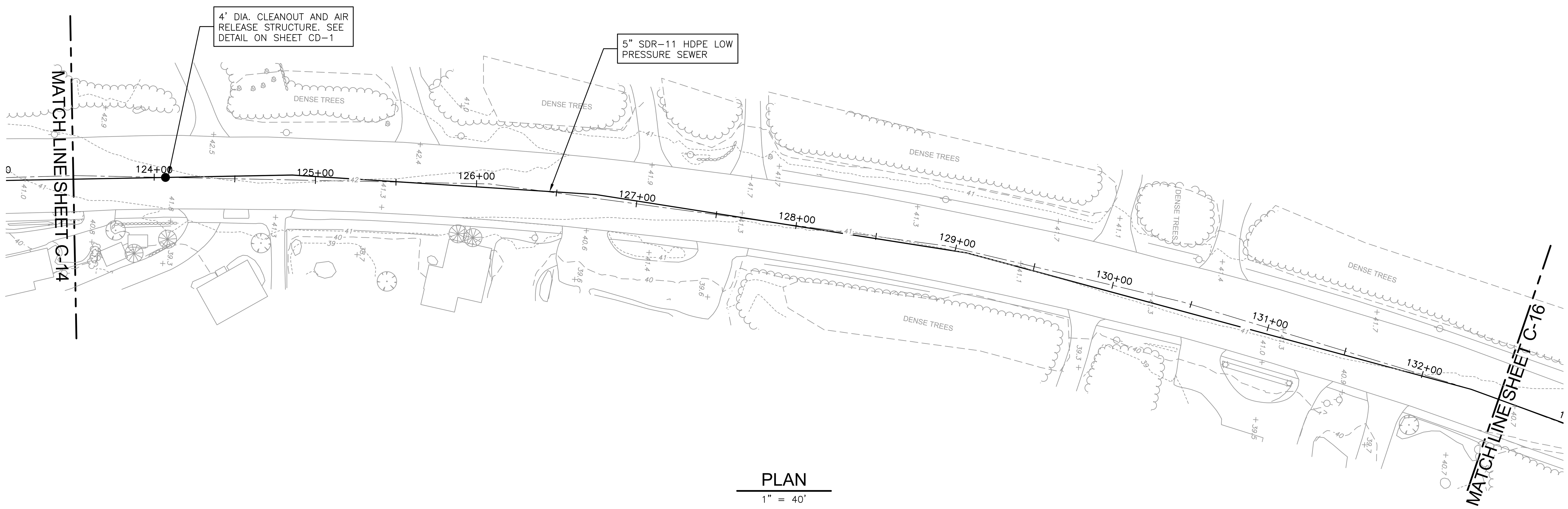
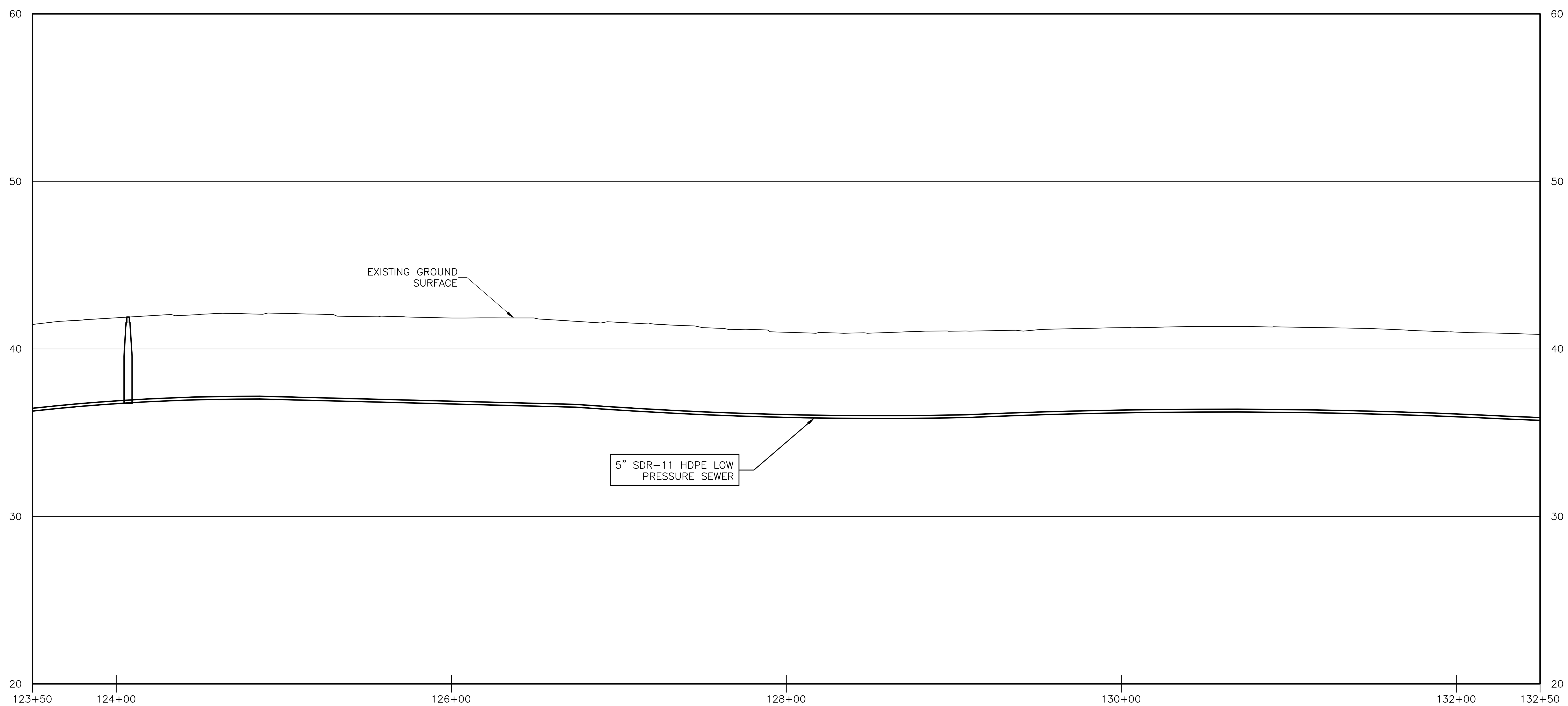
MILL STREET (ROUTE 6)



PLAN
1" = 40'



REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: J. HERMAN
DRAWN BY: L. KERSHAW
SHEET CHK'D BY: M. GUIDICE
CROSS CHK'D BY: C. KERSHAW
APPROVED BY: M. GUIDICE
DATE: JULY 2017

CDM Smith
260 West Exchange Street, Suite 300
Providence, RI 02903
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TOWN OF MARION, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

MILL STREET (ROUTE 6) IV

PROJECT NO. 0357-114884
FILE NAME: C015STPL.DWG
SHEET NO.
C-15

50% SUBMITTAL - NOT FOR CONSTRUCTION

XREFs: [CDMS_2436, 201611001WS3, CSTBP001, 201611002WS MARION] Images: []
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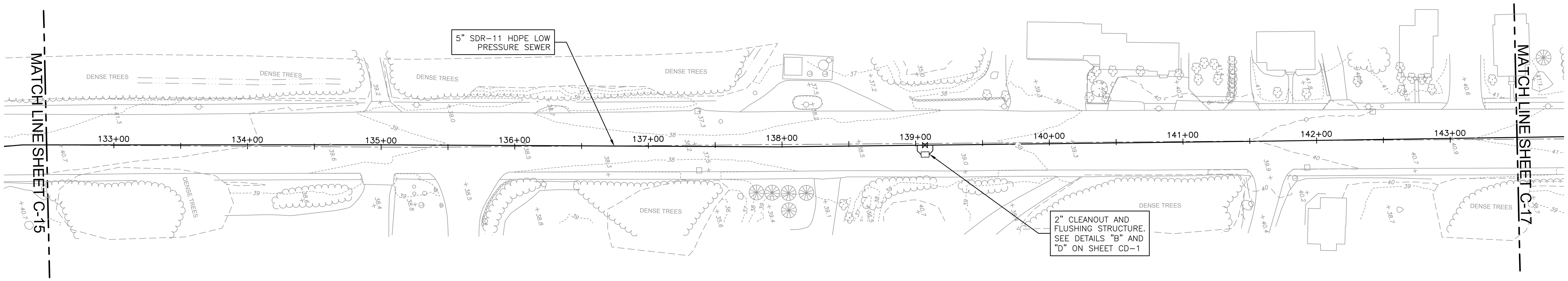
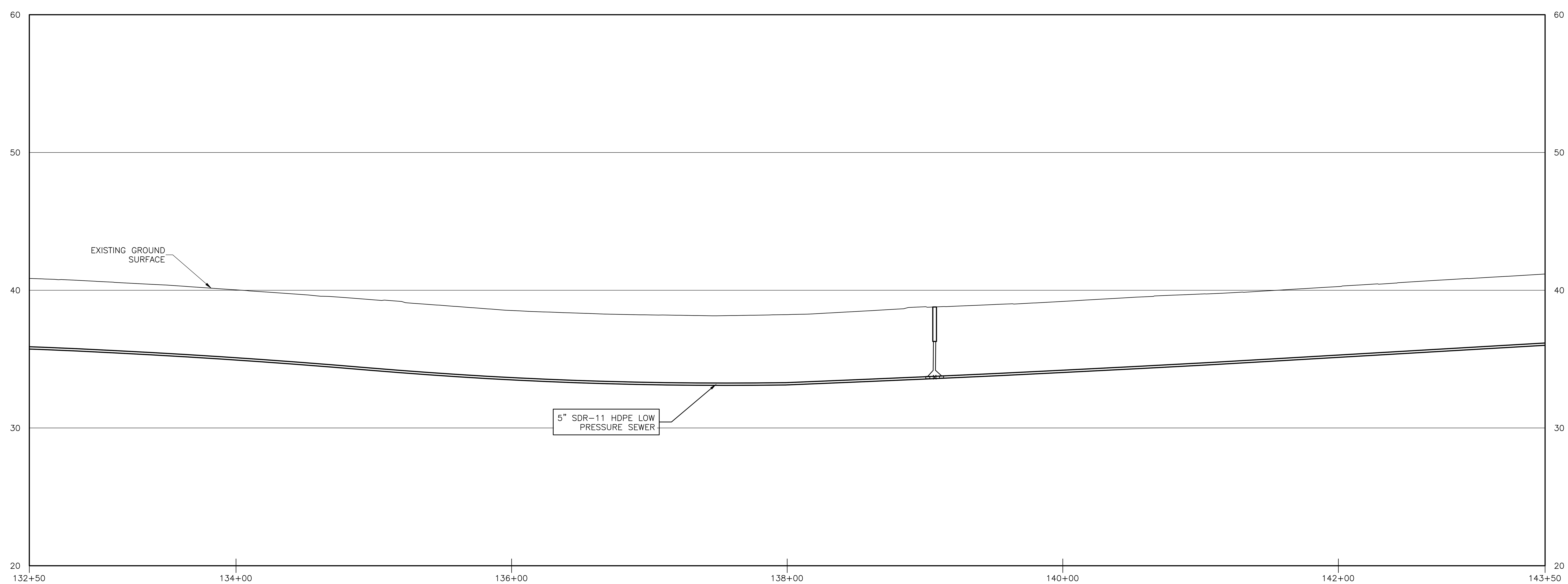
MILL STREET (ROUTE 6)



PLAN
1" = 40'



REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: J. HERMAN
DRAWN BY: L. KERSHAW
SHEET CHK'D BY: M. GUIDICE
CROSS CHK'D BY: C. KERSHAW
APPROVED BY: M. GUIDICE
DATE: JULY 2017

CDM Smith
260 West Exchange Street, Suite 300
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Tel: (401) 751-5360

TOWN OF MARION, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

MILL STREET (ROUTE 6) V

PROJECT NO. 0357-114884
FILE NAME: C016STPL.DWG
SHEET NO.
C-16

50% SUBMITTAL - NOT FOR CONSTRUCTION

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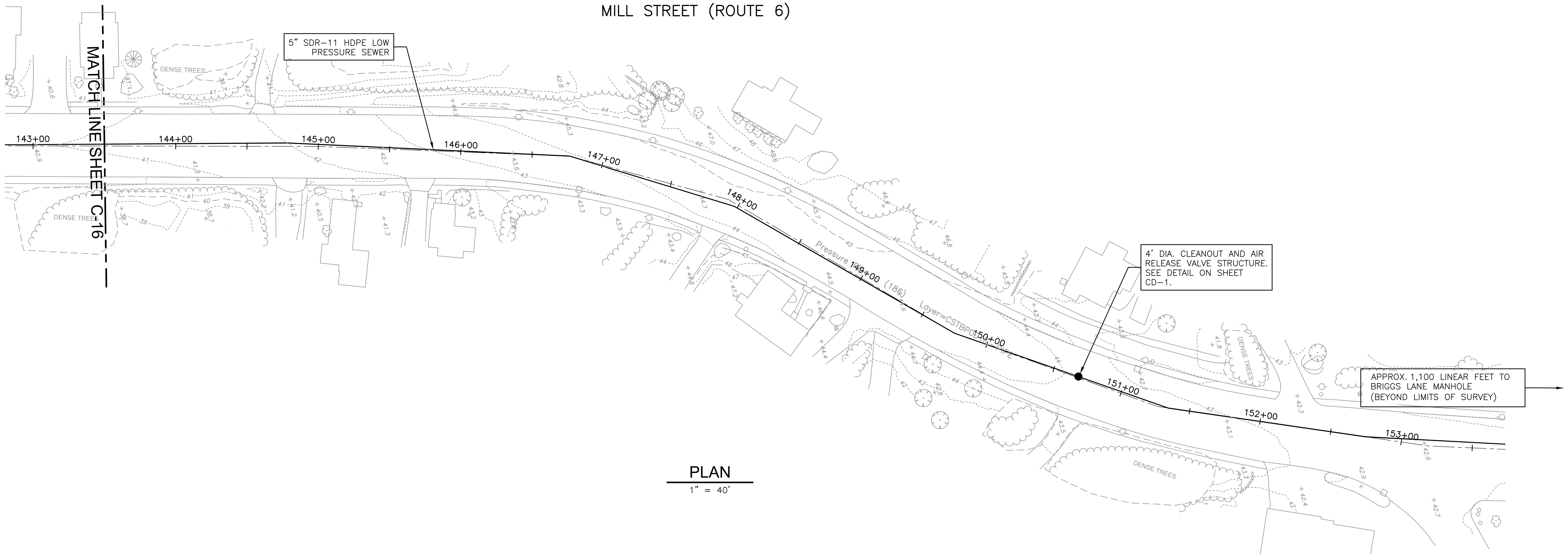
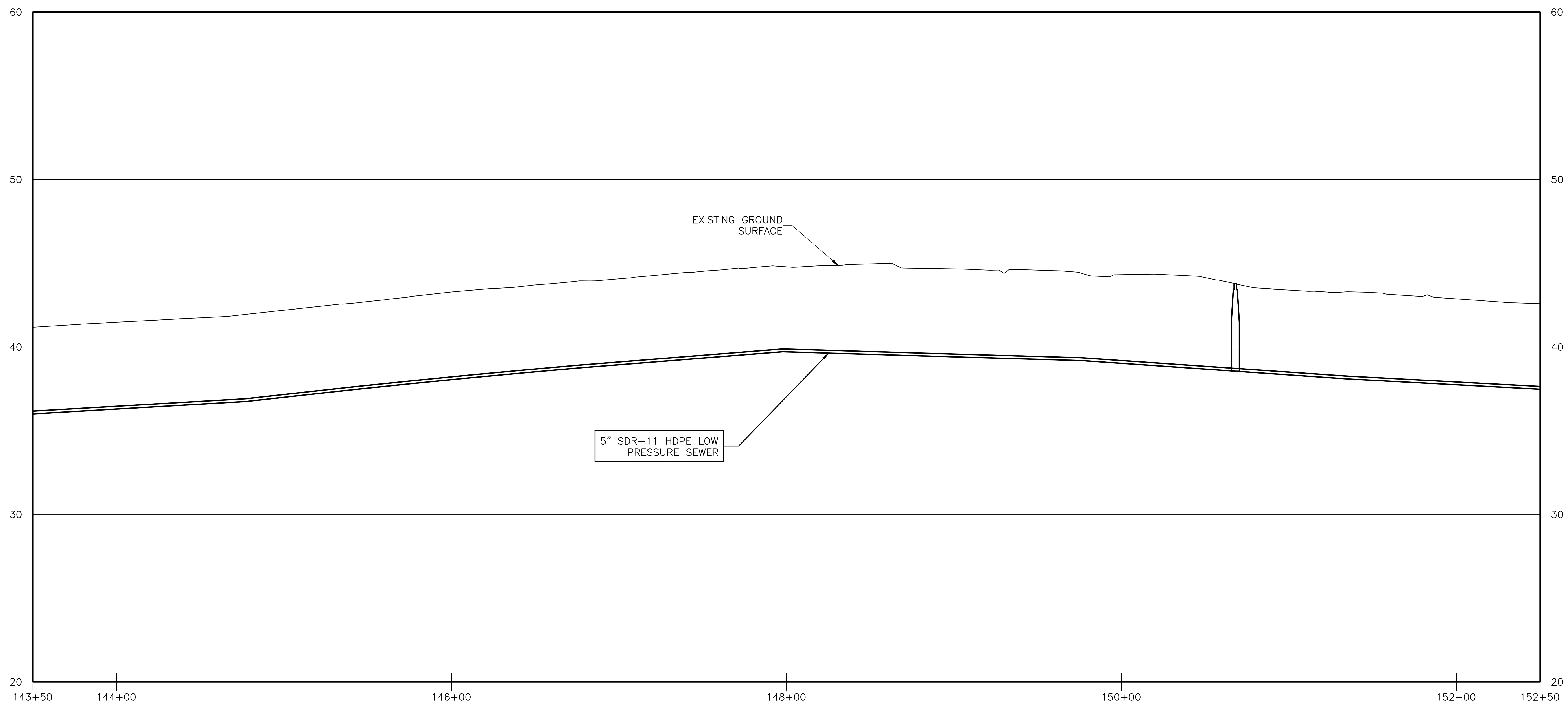


PLAN
1" = 40'



REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: J. HERMAN
DRAWN BY: L. KERSHAW
SHEET CHK'D BY: M. GUIDICE
CROSS CHK'D BY: C. KERSHAW
APPROVED BY: M. GUIDICE
DATE: JULY 2017

CDM Smith
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Tel: (401) 751-5360

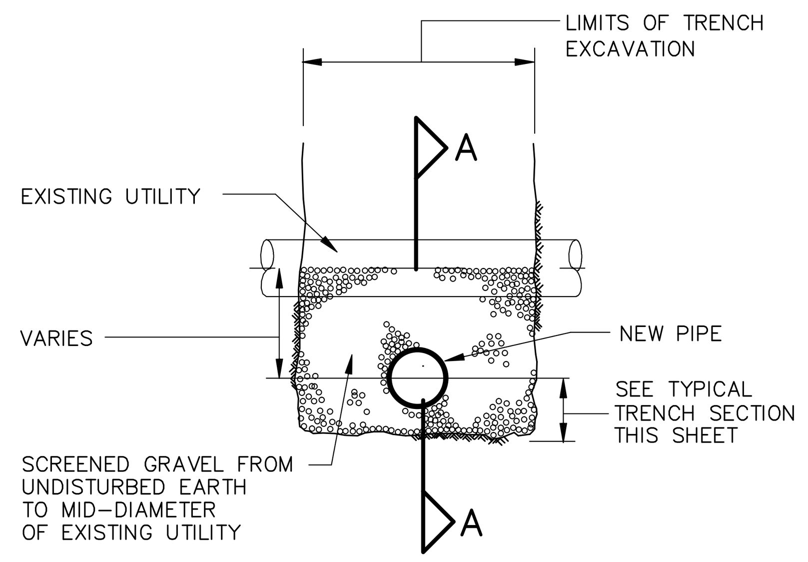
TOWN OF MARION, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

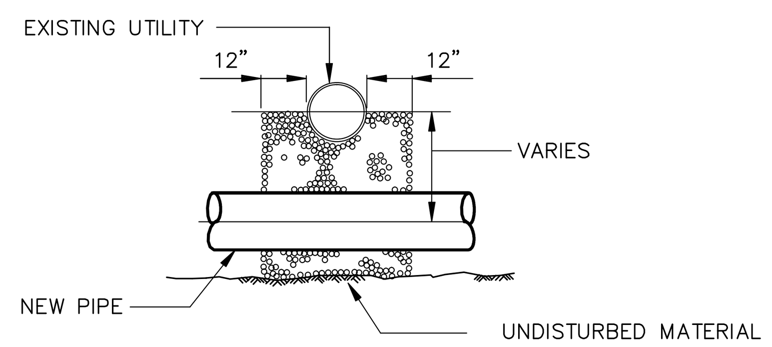
MILL STREET (ROUTE 6) VI

PROJECT NO. 0357-114884
FILE NAME: C017STPL.DWG
SHEET NO.
C-17

50% SUBMITTAL - NOT FOR CONSTRUCTION



TYPICAL SECTION

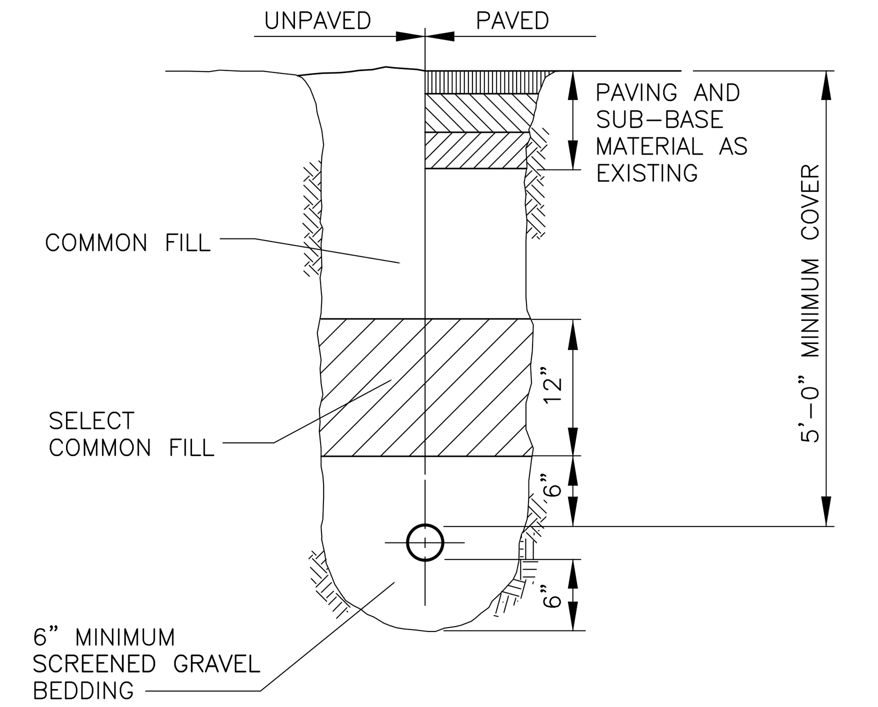


SECT A–A

UTILITY CROSSING DETAIL

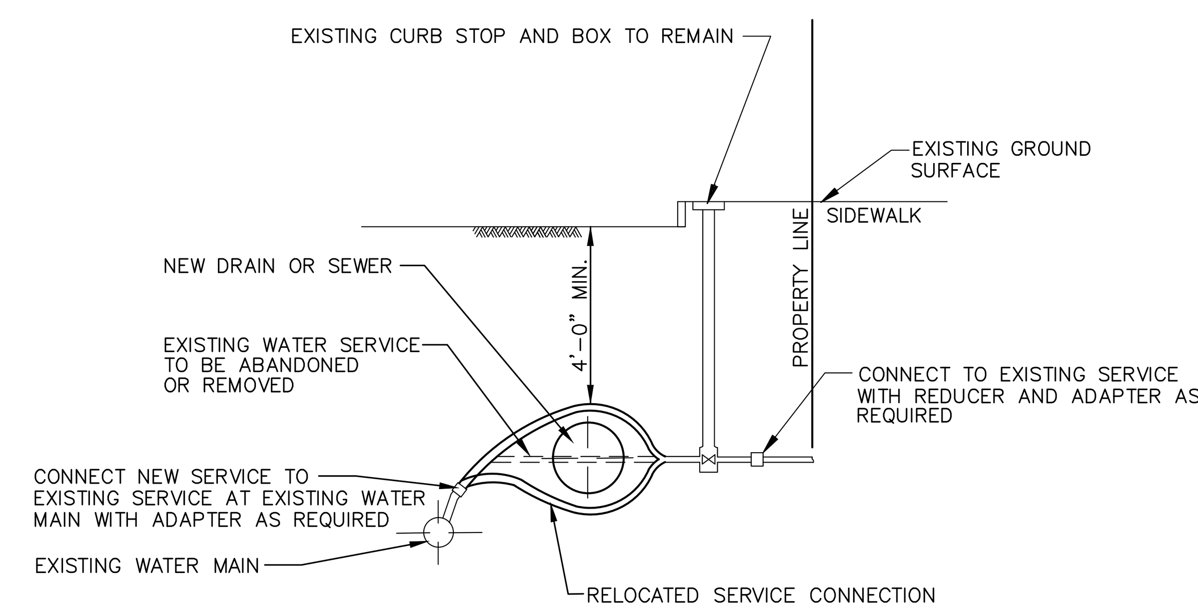
NOT TO SCALE

NOTE:
WHEN THE HORIZONTAL SEPARATION BETWEEN THE NEW SEWER AND THE EXISTING OR RELOCATED WATER MAIN IS LESS THAN 10–FEET OR THE VERTICAL SEPARATION IS LESS THAN 18–INCHES, THE WATER MAIN SHALL BE EXPOSED AND ENCASED IN CONCRETE FOR A MINIMUM OF 10–FEET ON EACH SIDE OF THE WATER MAIN CENTERLINE.



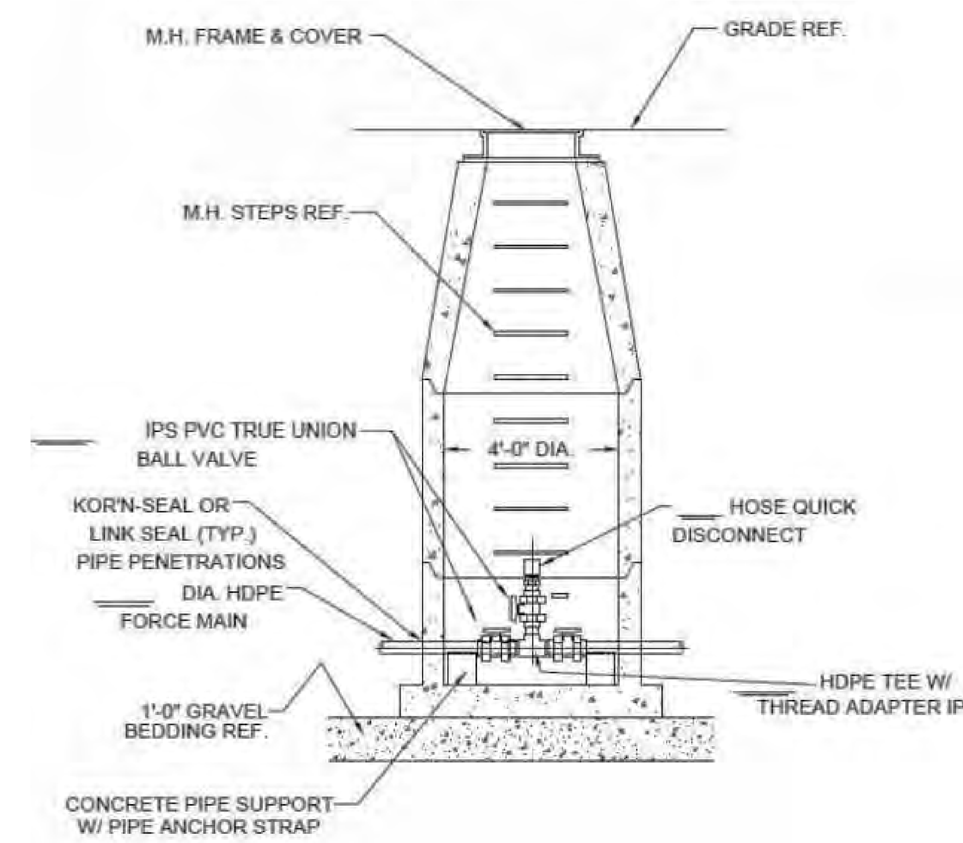
TYPICAL TRENCH DETAIL

NOT TO SCALE



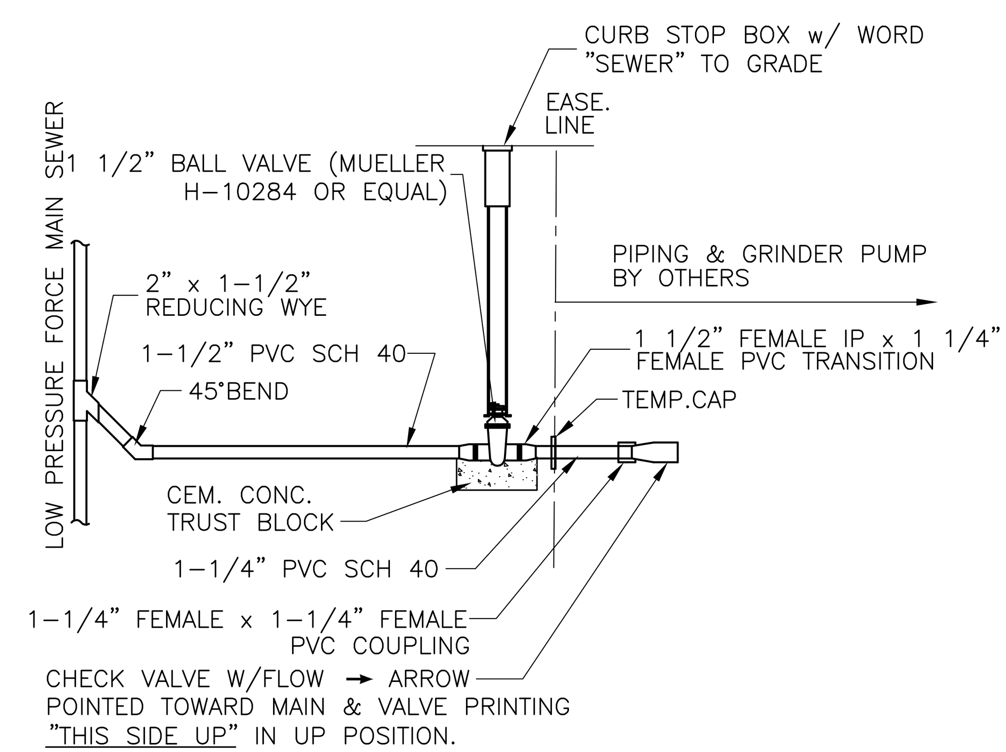
WATER SERVICE RELOCATION DETAIL

NOT TO SCALE



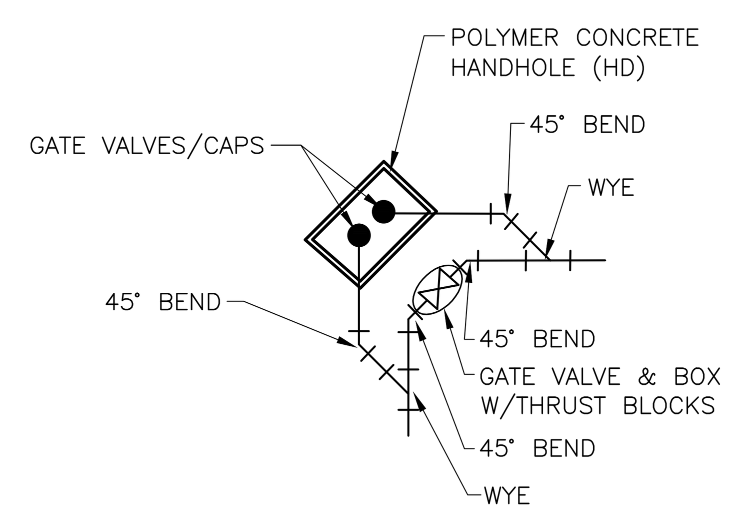
AIR RELEASE VALVE STRUCTURE

NOT TO SCALE

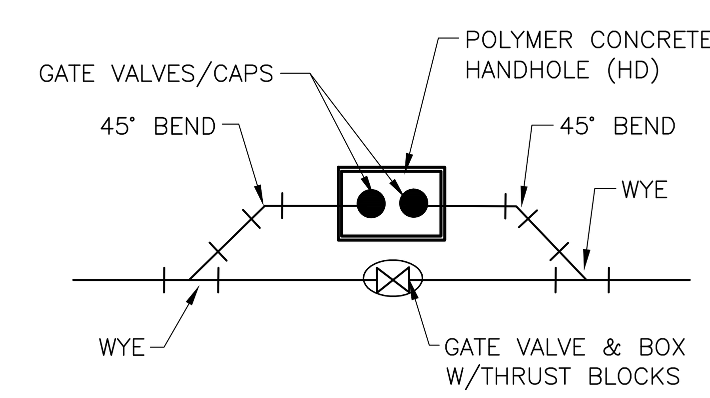


LOW PRESURE SEWER CURB STOP

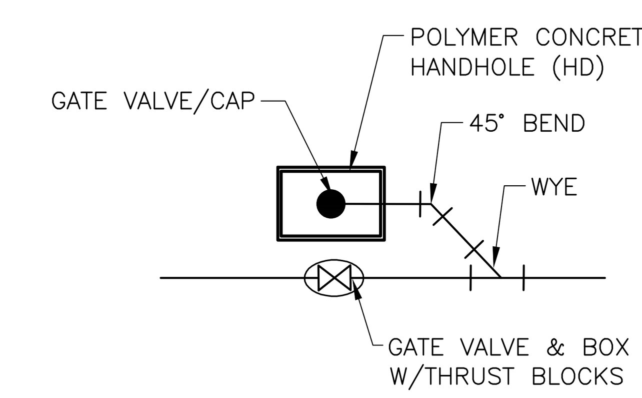
NOT TO SCALE



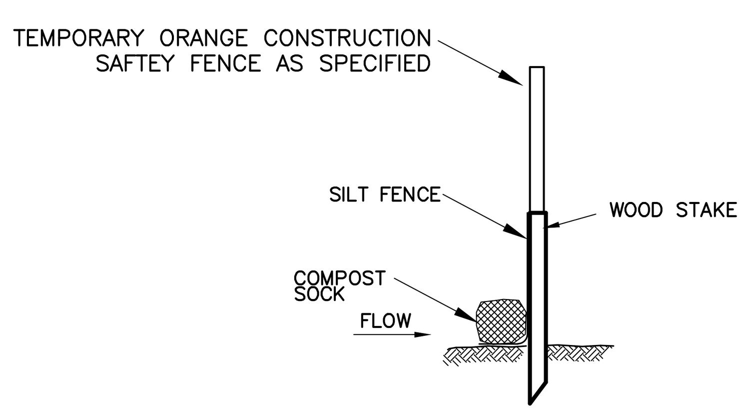
DETAIL "A"
CORNER CLEAN OUT PLAN



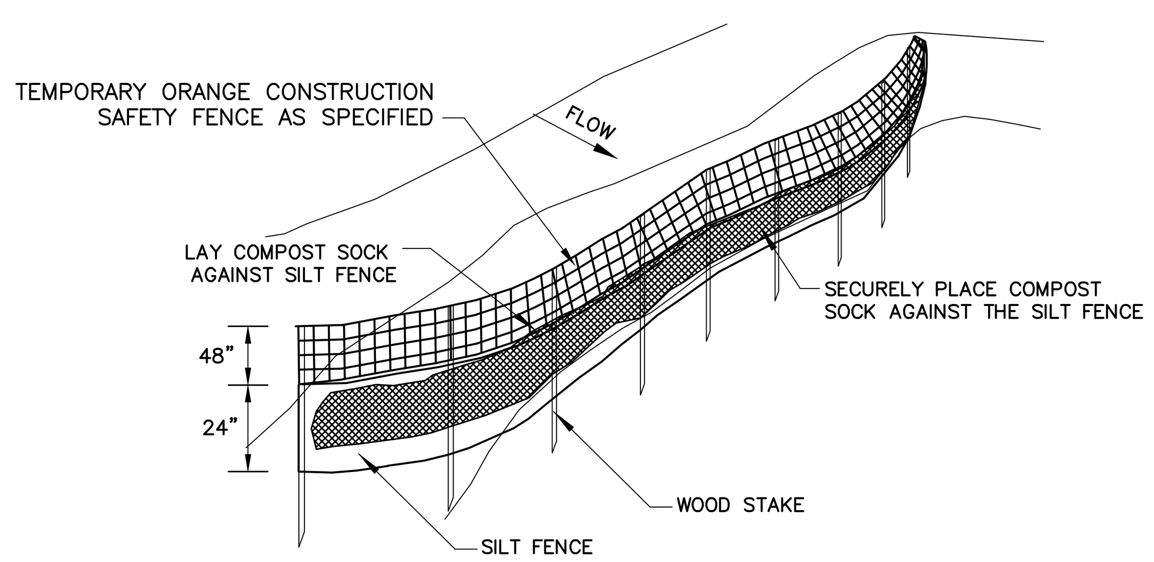
DETAIL "B"
IN LINE CLEAN OUT PLAN



DETAIL "C"
CLEAN OUT NEAR END OF LINE PLAN



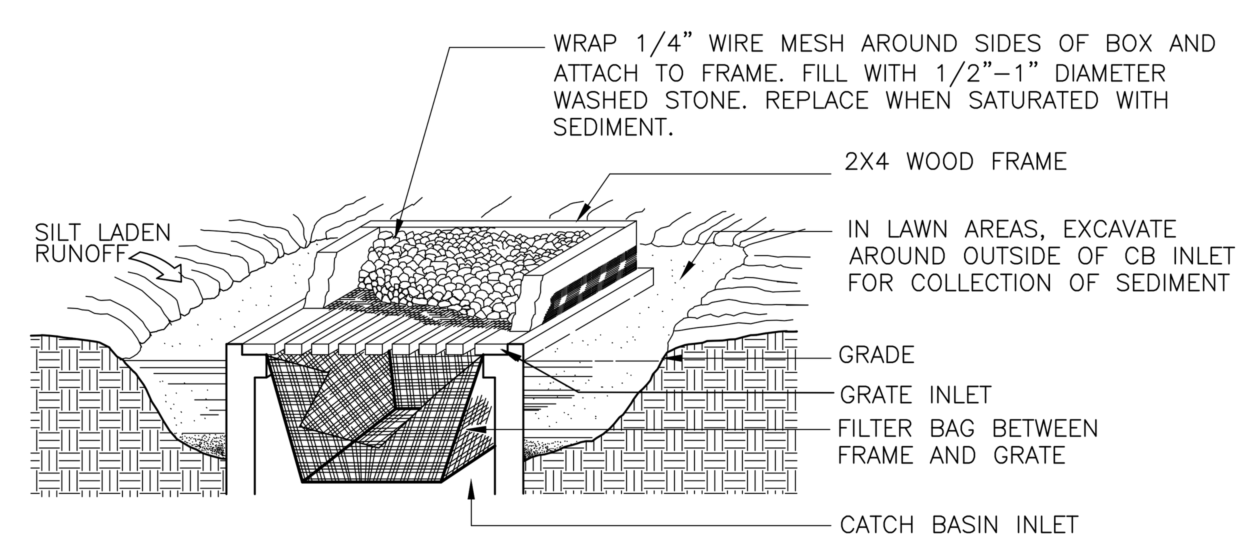
COMPOST SOCK
PLACEMENT DETAIL



PLACEMENT AND ANCHORING DETAIL
SILT FENCE AND COMPOST SOCK

ANCHORING FOR SILT FENCE AND SILT SOCK

NOT TO SCALE

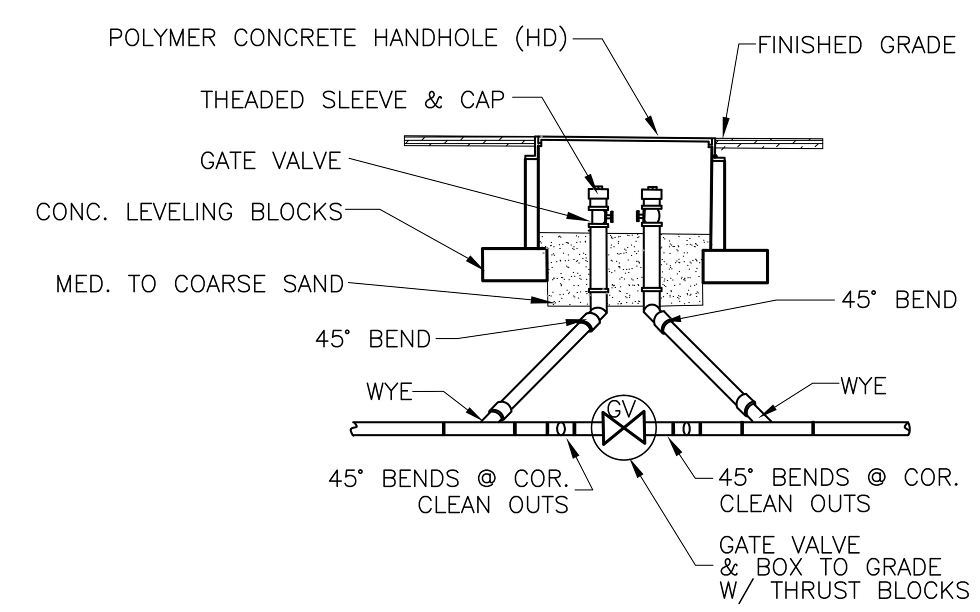


CATCH BASIN FILTER

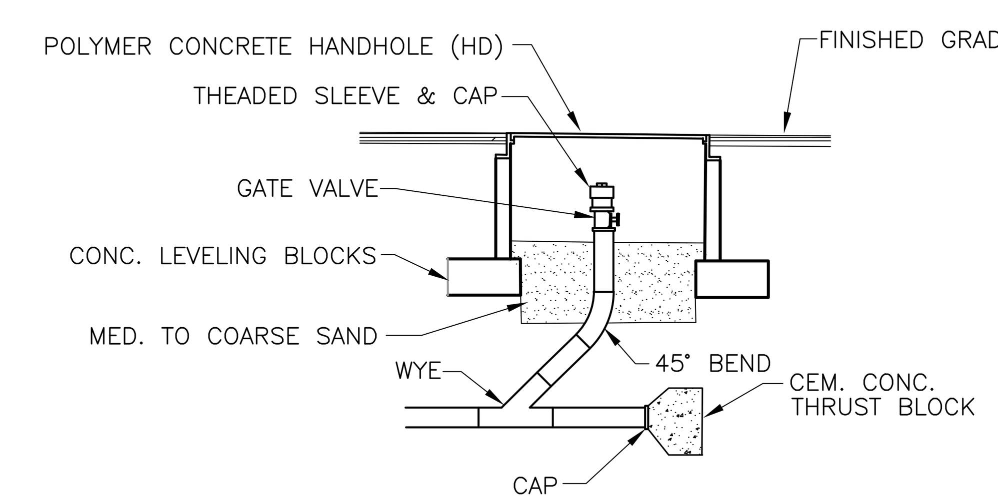
NOT TO SCALE

NOTE:
1) IN LAWN AREAS, USE MESH, WOOD FRAME BOX, AND STONE FOR INLET PROTECTION (NO BAG)

2) IN PAVED AREAS, USE FILTER BAG FOR INLET PROTECTION (NO MESH, BOX, OR STONE)



DETAIL "D"



DETAIL "E"

CLEAN OUT & FLUSHING STRUCTURE

NOT TO SCALE

REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY:	J. HERMAN
DRAWN BY:	J. HERMAN
SHEET CHK'D BY:	M. GUIDICE
CROSS CHK'D BY:	C. KERSHAW
APPROVED BY:	M. GUIDICE
DATE:	JULY 2017

CDM Smith
260 West Exchange Street, Suite 300
Providence, RI 02903
Tel: (401) 751-5360

TOWN OF MARION, MASSACHUSETTS

AUCOOT COVE SEWER EVALUATION

MISCELLANEOUS DETAILS

PROJECT NO. 0357–114884
FILE NAME: CD01STDT.DWG
SHEET NO.
CD-1

50% SUBMITTAL - NOT FOR CONSTRUCTION