Section 3
Sewer System Planning

3.1 Introduction

This section describes the criterion that was used for evaluating various alternatives for sewering the project area. Also, in an effort to better define sewer options, CDM completed a limited test boring program, windshield surveys, site walks, preliminary wetlands assessment, and a Phase IA archaeological survey as part of evaluating available conceptual layouts. These field programs were used to identify areas of ledge and high groundwater, assess possible sewer pipe routes, verify the existence of buildings, assess possible locations of pumping stations, and consider future permitting that may be required. The information gathered as part of these field programs is the foundation of all subsequent alternative analysis, including the development of cost estimates.

3.2 Sewer System Planning Criteria

Several factors are considered during the preliminary planning of wastewater conveying systems including: topography (maximize the use of gravity sewers), future flow estimates (used to size pipelines), 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.).

3.2.1 General

To operate a municipal wastewater system, wastewater must be collected from individual buildings and conveyed to the wastewater treatment plant. Collection systems are most frequently comprised of conventional gravity sewers, although topography and depth to bedrock generally dictate the sewer type and layout (i.e., conventional gravity sewers, low-pressure sewers, and force mains).

Various methods were used to choose locations of proposed sewers and force mains. Conceptual layouts were prepared on topographic maps followed by CDM conducting field investigations to help confirm the layout of the sewer pipe, identify properties that may or may not have existed at the time of City's aerial survey, identify properties that cannot be served by gravity in a cost-effective manner, and identify alignments for stream river crossings.

Where applicable, the layouts of street and cross-country sewers were planned in sufficient detail so that downstream facilities could be installed at elevations deep enough to serve a maximum amount of upstream tributary areas. If certain portions of the system are to be installed by private developers, the City should ensure that sewers are the proper depth, diameter, and slope to serve the entire area contributing to them.

3.2.2 Gravity Sewers

Being as the majority of the Cohas Brook Interceptor has already been built, the remaining lateral sewers off of the interceptor in the project area will all be between 8-inch to 24-inch pipes. NHDES' New Hampshire Code of Administrative Rules (Enw-Wq 704.05) discusses six pipe materials that are approved for gravity sewer pipe: solid wall polyvinyl chloride (PVC), ribbed wall PVC, recycled PVC, ductile iron (DI) pipe, concrete (RC) pipe, and prestressed concrete cylinder pipe (PCCP). RC and PCCP are typically not used for small diameter sewers. PVC and DI pipe are commonly accepted in the industry as the most appropriate pipe materials for this application and range of diameters. PVC should generally be used for most applications for this project due to its lighter weight, ease of construction, and typically lower capital and installation cost. The New Hampshire Code of Administrative Rules requires the use of SDR 35 for PVC pipe diameters from 8-inch through 15-inch and either solid wall T-1 and T-2 or ribbed wall for PVC pipe diameters from 18-inch though 24-inch. DI pipe should be used if determined appropriate to meet NHDES standards for lack of ground cover or close proximity to water mains. There are many other types of pipe materials that can be considered for sewer conveyance, yet these materials are not on the approved list of NHDES pipe materials and have typically not been considered for use in Manchester due to cost and other factors. Selection of pipe material for each contract shall be determined during final design in accordance with these criteria, NHDES and City standards, and actual design conditions.

Sewer diameters were selected based on anticipated future flow carried in pipes with minimum design slopes. These slopes ensure a cleaning velocity greater than two feet per second when the sewer is flowing at least half full to prevent settling of solids in the pipeline. In order to ensure that proposed sewers are capable of transporting peak flows with a factor of safety for any unexpected conditions, the sewer pipe diameters have been selected to transport these flows while flowing less than completely full. For residential sewers (8-inch through 12-inch) the pipe diameters have been selected based on a depth criteria of flowing ½ full at the design capacity. The flow-depth criteria increases to ¾ full as the pipe sizes increase. Table 3-1 shows the standard design factors that were used.

Pipe Diameter (Inches)	Design Flow/Full Pipe Flow (%)	Minimum Design Slope (ft/ft)
8	50	0.0040
10	50	0.0028
12	50	0.0022
15	60	0.0015
18	70	0.0012
21	75	0.0010
24	75	0.0008

Table 3-1
Pipeline Capacity Design Factor for Sizing Pipelines

3.2.3 Criteria for Sewer Depths and Chimneys

At a minimum, the top of the proposed sewer main should be approximately 6-feet below the foundation sill elevation. This elevation should allow for a gravity sewer service serving the first floor of the house. For homes with a full basement, considerations should be given for the top of the proposed sewer main to be approximately 10-feet below the foundation sill elevation. This elevation should allow for a gravity sewer service serving the basement floor. Attempts should be made during design to review septic system tie card information, if available, from Manchester's Board of Health and to meet with property owners that are critical to the depth of the sewer main to reduce the depth of sewer where possible. The overall depth of the sewer main downstream and subsurface conditions (i.e., rock excavation) required to meeting the criteria also needs to be considered when determining the appropriate depth. During design, a cost analysis should be conducted to determine if the additional cost for providing gravity service to the basement floor homes with a full basement is warranted versus providing these homes with an individual grinder pump or gravity service for the first floor only.

When the crown of the proposed gravity sewer is more than 13-feet below the foundation sill, a ductile iron chimney should be considered. A chimney is a tee connection that connects to the sewer system and extends vertically perpendicular to the sewer main. The purpose of the chimney is to decrease the depth of the sewer service lateral. Considerations for the design of a chimney should include:

- maintaining a depth of 6-feet below the road surface;
- allowing for the sewer service to be installed under the existing water main;
- extending the chimney to approximately 10-feet below the foundation sill for homes with full basements; and
- extending the chimney to approximately 6-feet below the foundation sill for homes without full basements.

The final elevation of all proposed sewer mains and potential use of chimneys should be confirmed during the final design and construction phases and should be evaluated on a case-by-case basis and discussed directly with the city.

3.2.4 Sewer Manholes

Precast concrete is commonly accepted in the industry as the most appropriate sewer manhole material. The exterior surface of the sewer manhole should be coated with two coats of bituminous waterproofing material. Being as the anticipated sewer pipes for this project are 24-inch and smaller, most sewer manholes should have a 4-foot inside diameter. Sewer manholes with internal drop connections should have a 5-foot inside diameter. The manholes should be tested for leakage in accordance with industry standards. The invert of the sewer manholes should be constructed with brick masonry. Sewer manholes 15-feet and under should be installed without rungs.

Sewer manholes over 15-feet should be installed with rungs and sewer manholes over 20-feet should be installed with a platform at the vertical midpoint of the manhole (i.e., the platform should be installed at about 11-feet below the road surface for a 22-foot deep manhole). For pipes under 48-inch diameter, the NHDES' standard states that the distance between manholes shall not be greater than 500-feet. The current City of Manchester standard is that the distance between manholes shall not be greater than 300-feet. With new sewer cleaning techniques, the City is currently revising this standard to allow for sewer pipe runs between manholes to be up to 400-feet. All sewer manholes should be designed and constructed per the City of Manchester and NHDES standards.

The manhole frame and cover must meet the City and State standard and have a 30-inch opening. The cover should have the word "SEWER" cast in 3-inch letters and "CITY OF MANCHESTER" and the year build (i.e., "2009") cast in 1-5/8-inch letters.

3.2.5 Low-Lying Areas

Certain areas of the Cohas Brook wastewater collection system are located in a low-lying area where gravity flow to the desired location is not possible. In these areas two alternatives to gravity sewers were considered, low-pressure sewers and gravity sewers with a pump station and force main. Each alternative can be appropriate given conditions where all homes cannot be served by gravity in a cost-effective manner.

Low-Pressure Sewers/Grinder Pumps

If an area of homes to be provided sewer service is small (generally less than 30 homes) or involves large changes in topography, individual private grinder pumps with a low-pressure sewer system can be an effective means to provide collection. Appendix F provides information on low pressure sewer systems and grinder pumps.

A grinder pump grinds up wastewater solids produced in a home (i.e., toilet use, shower, and washing machine) and pumps it into the public sewer system. A grinder pump is placed in a tank (or well) that is buried in an outdoor location on a homeowner's property (grinder pump units also can be installed inside a home). The tank provides wastewater storage capacity and, when the level in the tank reaches a pre-set value, the grinder pump automatically turns on, grinds the waste, and pumps the waste out of the tank through the homeowner's sewer service line and into the City's sewer system. The pump is powered by electricity and will normally run for one or two minutes, and automatically turns off when the tank is emptied.

Low-pressure sewers are generally 4-inches in diameter or less. The pressure sewers and fittings are often PVC pipe that are rated for the expected internal pressure (i.e., class SDR 21) The slope of the sewer is not important since the system is pressurized, which allows the pipes to follow the natural topography of the land. Because of the small diameter of the pipeline and the shallow pipe installation, these sewers are generally less expensive to construct than conventional gravity sewers with pumping stations. There are several neighborhoods in the project area where low-pressure

sewers were considered and recommended to provide service (See alternatives analysis in Section 4).

In some cases, it is necessary to install individual grinder pumps to provide service to homes that lie significantly below the existing road grade. This option allows the gravity sewer to be installed at a higher elevation than would be required to service all homes entirely by gravity, thereby reducing project cost. Homes with individual grinder pumps simply pump up to and connect to the new gravity main in the street. There are several homes in the project area that will be required to pump their sewer up to the gravity main in the street.

Pump Stations/Force Mains

If an area of homes to be provided sewer is approximately 30 homes or more and the topography generally slopes in the same direction to a low point, gravity sewers can be installed to collect the local sewer and convey the flow to a pump station at the low point. The pump station is used to "lift" the wastewater through a force main to a location with a higher elevation where the wastewater can resume gravity flow toward the desired destination.

Local sewers to the pump station are sized like a typical gravity sewer. The pump stations are designed to handle the expected peak hour wastewater flow that is tributary to the station. Pump stations are typically designed to have a minimum of two pumps, with the capability of pumping the entire design flow with one pump out of service, for redundancy. Usually pump stations are supplied with an emergency power generator, which would automatically operate in the event of a power failure. In recent years, the City has generally used Gorman-Rupp suction lift pumps in their stations.

The force main leaving the station is sized to maintain a minimum velocity of 3 feet per second to prevent debris from accumulating, while minimizing head losses and thereby reducing energy costs. Typically, the minimum pipe diameter for force mains is 4 inches. Force main sewers in New England are generally Class 52 ductile iron pipe with an interior cement mortar lining and asphaltic seal coat an exterior bituminous outside coating. However, Manchester Water Works has noted that recently installed ductile iron pipe in the project area has been corroding due to existing soil conditions. Although pump stations and force mains were considered in some locations, none were recommended in the proposed sewer layouts (See alternatives analysis in Section 7). Therefore, considerations for alternative pipe material did not need to be considered further.

3.3 Preliminary Geotechnical Exploration Program

3.3.1 Previous Subsurface Exploration Program

Geotechnical exploration programs were previously conducted by others in the project area. The boring logs and boring locations were provided to CDM by the State of New Hampshire Department of Transportation and the City of Manchester, NH and are summarized below. Relevant previous test boring logs are provided in Appendix B. Previous test boring locations are shown on Figure 1 in Appendix B.

State of New Hampshire Department of Transportation
A subsurface exploration program was previously conducted by the State of New Hampshire Department of Public Works and Highways between February 2000 and July 2001 for constructing overhead traffic signs on Route 101 and the Interstate 93 Exit 6 and Candia Road construction. Thirteen test borings were drilled to depths ranging between 10 and 30 feet.

Goldberg – Zoino & Associates Inc.
A subsurface exploration program was previously conducted by Goldberg – Zoino & Associates Inc. (GZA) of Manchester, NH between April 1982 and August 1982 for the City of Manchester. Six test borings were drilled to depths ranging between 11 to 27.5 feet.

Miller Engineering & Testing Inc.
A subsurface exploration program was previously conducted by Miller Engineering & Testing, Inc. of Manchester, NH between May 2005 and August 2005 for design and construction of the Phase II Interceptor. Twelve test borings were drilled to depths ranging between 9.4 to 40 feet.

3.3.2 Recent Subsurface Exploration Program

The purpose of this preliminary geotechnical exploration program was to investigate the subsurface conditions within the general area of proposed sewers to aid in the preliminary design of the project. The depth and location of the subsurface explorations was based on the preliminary layout of all pipe.

Fifteen test borings, B-1, B-2, B-4, B-6 through B-15, and B-18, were drilled by New Hampshire Boring, Inc. of Londonderry, New Hampshire (NHB) between September 17, 2008 and September 24, 2008. Five test borings B-3 and B-21 through B-24, were drilled by NHB between November 19, 2008 and November 21, 2008.

Test boring B-3 was conducted with an all-terrain vehicle (ATV)-mounted drill rig. The remaining test borings were conducted with a truck-mounted drill rig. Three borings, B-1, B-8, and B-11, were conducted using 2-$^{3}/_{4}$ inch inside diameter Hollow Stem Augers (HSA). The remaining borings were conducted using flush jointed casing with drive and wash techniques. Split spoon samples were typically taken in the test borings at 5 foot intervals over the total depth of each boring. The samples were obtained in accordance with ASTM D1586 (2 inch outside diameter split spoon

sample, driven 24 inches by blows from a 140 pound hammer free falling 30 inches). The number of blows required to drive the sampler each 6 inch increment was recorded and the Standard Penetration Resistance (SPT) N-value was determined as the sum of the blows over the middle two increments. Representative soil samples from each split spoon were collected, logged, and stored in jars for later review.

Rock coring was conducted at borings B-3, B-4, B-6, B-7, B-10, B-13, B-21, and B-22 using an NX core barrel in accordance with to ASTM D2113. The percent recovered and rock quality designation (RQD) was recorded in the lab by a CDM representative after obtaining the samples from NHB. The RQD was determined for each core run by dividing the total length of the rock segments longer than four inches over the total length of the core run. The time to advance each foot of rock core was recorded during the rock coring process by the drillers. Rock core samples from each boring were stored in rock core boxes for later review.

The test boring depths ranged from 12 to 41 feet below ground surface. Groundwater levels at the test borings were recorded by the drillers upon completion of the boring exploration. Borings conducted using drive and wash techniques may not accurately reflect static groundwater levels. Each borehole was backfilled with drill cuttings and sand upon completion. Asphalt patch was placed at the ground surface where boreholes were conducted within existing paved areas.

The borings were located in the field by taping and line of sight from existing site features. The as-drilled locations are shown on Figure 1 in Appendix B, and the recent test boring logs are included in Appendix B.

3.3.3 Subsurface and Groundwater Conditions

Subsurface Conditions

In general, subsurface conditions encountered during the recent test boring program consisted of a miscellaneous granular fill, underlain by a silt layer with varying amounts of sand, followed by a sand layer with varying amounts of silt. The sand layer was underlain by gravel and weathered rock layers before encountering either Bedrock or refusal. Refusal was encountered in borings B-1 through B-7, B-9, B-10, B-13, B-14, B-21, and B-22 and is defined as less than 6 inches of penetration for 100 blows from a 140 pound hammer. The depths at which refusal was encountered may or may not indicate the presence of bedrock. A summary of the subsurface conditions encountered at the test boring locations is presented on Table 1 in Appendix B.

Asphalt pavement was encountered at all recent borings except test boring B-3 and ranged in thickness between 1 and 9 inches. The thickness of the asphalt was not recorded in test borings B-4 or B-11.

Typically, the fill consisted of dry to wet, loose to very dense, brown, fine to coarse sand, with varying amounts of silt and gravel. The fill layer ranged between 2.7 and 8.5 feet in thickness and SPT N-values typically ranged from 18 to 46 blows per foot

(bl/ft) at the recent exploration locations. Fill was encountered below the asphalt pavement at all locations.

A naturally deposited silt layer was encountered below the fill layer at test boring locations B-2, B-3, B-9, B-22, and B-24. It typically consisted of wet, very stiff to hard gray silt, with varying amounts of sand and gravel. In borings B-2 and B-3, silt and clay was encountered underlying the fill layer. These layers ranged between 2.5 and 22 feet in thickness and SPT N-Values typically ranged from 20 to 60 bl/ft at the recent exploration locations.

The sand layer underlying the silt layer was encountered at all recent boring locations except B-3, B-4, B-6, and B-21. It typically consisted of wet, medium dense to very dense, brown coarse to fine sand with varying amounts of silt and gravel. At test borings B-7, B-10, B-13, and B-22 the sand layer was fully penetrated and ranged in thickness between 1.9 to 5.0 feet. In the remaining test borings where the sand layer was encountered but not fully penetrated, strata thickness ranged between 5.0 and greater than 37.0 feet. The SPT N-Values typically ranged from 20 to greater than 100 bl/ft at the exploration locations.

In test borings B-4, B-6, B-7, B-8, B-10, B-11, B-13, B-21, and B-22 weathered rock was encountered. Samples were not collected within this layer. The thickness of the strata is based on drilling conditions and drill cuttings. This weathered rock layer ranged between 0.5 to 3.8 feet in thickness.

Test borings B-3, B-4, B-6, B-7, B-10, B-13, B-21, and B-22 penetrated into the underlying bedrock. Bedrock may have been encountered at the bottom of borings B-1, B-2, B-9, and B-14 based on blow counts greater than 100 bl/6 inch intervals, but cores samples were not taken in these locations. Where sampled, the bedrock typically consisted of very hard, slightly fractured, fresh to moderately weathered, fine to coarse grained, gray and pink granite at depths ranging from 8 to 26 feet below ground surface. Hard, slightly fractured, freshly weathered, coarse, black/green dolerite was encountered in boring B-21. Rock cores were visually classified by a CDM representative after reviewing core samples in the CDM laboratory. The drill rates were recorded in the field and ranged between 2 and 9 minutes per foot. The RQD value for the rock cores were recorded in the lab and ranged from 32 to 88 percent, except RQD for the sample cored at B-1 was 0 percent.

Groundwater Conditions

Groundwater was encountered in all test borings except B-13. Groundwater depth ranged from water at ground surface to 24 feet below ground surface. Borings B-1, B-8, and B-11 were drilled using hollow stem augers, all other borings were drilled using drive and wash methods. The drive and wash drilling method introduces water into the boring hole, therefore, groundwater depth measurements may not accurately reflect actual conditions.

3.3.4 Variation in Subsurface Conditions

Interpretation of general subsurface soil conditions presented herein is based on soil, rock, and groundwater conditions observed during the test boring program. However, subsurface conditions may vary between exploration locations.

Groundwater levels change with time, season, temperature, and construction activities in the area, as well as other factors. In addition, stabilized groundwater levels can be difficult to obtain in borings drilled using drive and wash drilling methods due to the presence of drilling fluid in the borehole. Therefore, groundwater conditions at the time of construction may be different from those found during the time of the explorations.

Generally, there is no indication from review of the results of the limited soil boring program that the subsurface conditions are unsuitable for pipe installation. During the final design, additional subsurface explorations should be performed to identify the trenching and backfilling requirements to install the pipe, including trench support and braced excavation, rock excavation, dewatering, and pipe bedding requirements, including use of filter fabric, over excavation, suitability of excavate for use as trench backfill, potential contaminated soils, etc. Further geotechnical investigations required during the design phase are discussed in Section 7.2.7. Implementation considerations during the construction phase, including adhering to the City of Manchester's Blasting Permit, are also discussed in Section 7.

3.4 Historic and Archaeological Resources

Independent Archaeological Consulting, LLC, (IAC) completed a Phase IA archaeological sensitivity assessment (see Figures 1 through 5 in Appendix C) for the entire Cohas Brook Sewer Master Plan area. The purpose of the study was to follow up on a previous archaeological assessment completed in 1995 (Wheeler 1995). The archaeological assessment was authorized under Section 106 of the Historic Preservation Act of 1966 (P. L. 89-665), as amended, and implemented by regulations of the Advisory Council on Historic Preservation (36 CFR Part 800), as part of the state permitting process through the NHDES.

The purpose of the archaeological assessment was to identify known archaeological sites and to define the sensitivity of project areas. "Archaeological sensitivity" is a qualitative term referring to the relative probability that archaeological sites are present. If an area has high sensitivity, the likelihood is good that resources are present. If sensitivity is low, archaeologists consider it unlikely that area has archaeological resources. Moderate sensitive refers to a middle ground between the two ranks of high and low. The sensitivity assessment was completed through review of known archaeological resources as inventoried in the New Hampshire Division of Historic Resources (NHDHR) site files in Concord; cartographic analysis of landform, topography, soils, and proximity to water; review of secondary historic sources; and a walkover (inspection) survey.

For Native American or pre-Contact-period archaeological resources, areas that are close to water, on level terrain, and on well-drained soils have high sensitivity. These high sensitivity areas must also be relatively undisturbed by modern development. Where the same environmental conditions are met – but have some modern impacts – the area has a moderate sensitivity for archaeological resources. An area that is highly disturbed, steeply sloped, poorly-drained, or is more than 100 meters (m) (328 ft) away has low sensitivity for pre-Contact-period archaeological resources.

For Euroamerican archaeological resources, where features were portrayed on historic maps – such as dwellings, schools, churches, cemeteries, and mills – archaeological sensitivity is high. Areas remotely removed from transportation systems (i. e., more than 200 m (656 ft) away from roads, bridges, and railroads) have low sensitivity. Areas within 200 m (656 ft) of transportation systems but not shown on historic maps as having resources on them have a moderate sensitivity for Euroamerican archaeological resources.

Much of the cultural context for both Native American and Euroamerican resources was previously treated in the 1995 report. Since then, however, several transportation projects have garnered still more information about Native American settlement patterns relating to the Merrimack River drainage (e. g., Bunker 2003; King, Willan, and Mair 2001; Will and Mack 2008). Information on the location of new sites has been incorporated into the figures in Appendix C which show the project area of potential effect (APE).

Most of the proposed sewer pipes are recommended to be placed in road right-of-ways, which, for the most part, were evaluated with low sensitivity for archaeological resources. The exception to this is at road crossings that co-occur with the Cohas Brook drainage; road crossings that intersect any portion of the brook were evaluated as having moderate sensitivity. Along cross-country segments of the proposed sewer main, IAC considered the land use history, existing conditions, and degree of disturbance to evaluate whether archaeological resources might be present. All portions of the project area that intersect Cohas Brook are sensitive. Several other cross-country improvements traverse landforms where pre-Contact-period archaeological sites are known to exist, so any nearby effects are sensitive for Native American archaeological resources. The additional archaeological investigations that are required are summarized in Section 7.

3.5 Population and Wastewater Flow Estimates

Residential wastewater flows are a function of population. The 2007 Manchester population estimates according to the New Hampshire Office of Energy and Planning is 108,580 and the number of housing units is 48,618. Dividing the population by the number of houses gives an average of 2.2 people per house. The New Hampshire Code of Administrative Rules, Chapter Env-Wq 700 Standards of Design and Construction for Sewerage and Wastewater Treatment Facilities 704.03 (b) 2 states that an average daily per capita flow of not less than 70 gallons per day shall be

assumed. Equation 1 indicates how the average daily flow per household could be calculated:

[(2.2 people/house) x (70 gpd/person)] Equation 1

However, CDM recommends use of a more conservative 3.25 people per house because the number of bedrooms per house in this area is probably greater than the average for the city as a whole. Equation 2 indicates how the average daily flow per household was calculated:

[(3.25 people/house) x (70 gpd/person)] Equation 2

It is important to note that NHDES Administrative Rules and standard industry practice also require that, once the average daily flow is calculated, a peaking factor must be applied to account for periods of extreme high flows that may occur. Pump stations and treatment plant hydraulics are designed to handle the projected peak flow. To calculate the peaking factor, the Ratio of Extreme Flow to Average Daily Flow (from TR-16, Guides for the Design of Wastewater Treatment Works Prepared by the New England Interstate Water Pollution Control Commission), a tool that helps predict peak wastewater flow rates in New England Communities based on average daily flows, was utilized.

Infiltration from developed areas is assumed to be 300 gallons per inch diameter per mile per day according to the New Hampshire Code of Administrative Rules. Equation 3 indicates how infiltration could be calculated:

[(300 gpd) x (pipe diameter in inches) x (pipe length in miles)] Equation 3

However, CDM recommends use of a more conservative infiltration and inflow assumption for developed areas of 1,000 gallons per inch diameter per mile per day to account for unknown flows. Equation 4 indicates how infiltration from proposed sewer pipes was calculated for this project:

[(1,000 gpd) x (pipe diameter in inches) x (pipe length in miles)] Equation 4

To predict the future peak flow that will be delivered to the existing plant, the average daily flow for the entire City, including the proposed sewer extension, was calculated. When the effluent discharged for a period of 90 days exceeds 80 percent the City must submit to the permitting authorities loading projections up to the time when the capacity of the treatment facility will be reached, and a program for maintaining satisfactory treatment levels consistent with approved water quality management plans. The values shown on Table 3-2 indicate that the future flows delivered to the WWTP will not exceed 80 percent (27.2 MGD) of the 34 MGD design capacity.

Type of Flow	Manchester Flow (MGD)	Auburn Flow to Bypass 28 (MGD)	Auburn Flow to Bodwell Road (MGD)	Londonderry Flow to Bodwell Road (MGD)	Average Daily Flow (MGD)
Existing sewer flow[1]	23.1	0	0	0	23.1
Additional sewer flow from existing buildings in project area[2]	0.3	-	-	-	0.3
Potential additional flow from future development in project area[3]	0.7	-	-	-	0.7
Infiltration from 15.4 miles of proposed pipe in project area[4]	0.1	-	-	-	0.1
Anticipated flow from neighboring towns to the project area[5]	-	0.3	0.2	0.1	0.3
Total	24.2	0.3	0.2	0.1	24.5

1. Estimated existing sewage flow taken from the AECOM; May 2009 Draft Facility Plan Report – Chapter 2.
2. Additional sewage flow from buildings in project area was calculated with Equation 2.
3. Potential additional flow from future development is based on current Manchester zoning and Equation 2.
4. Infiltration from proposed new pipes was calculated using Equation 4.
5. Estimated sewage flow from Londonderry to Bodwell Road taken from "Wastewater Facilities Plan Update for the Town of Londonderry, New Hampshire"; Wright-Pierce; January 2005. Estimated sewage flow from Auburn to Bodwell Road taken from email provided by City of Manchester on December 10, 2008 and to Bypass 28 was taken from May 2009 City of Manchester comments on draft report.

Table 3-2
Projected Future Wastewater Flow with Project Implementation

3.6 Sewer Unit Cost Development

Project unit costs for the Cohas Brook Sewer Project were developed based on a review of public construction cost bid results from a number of recent projects in New Hampshire and northern Massachusetts. From these bid results, typical costs were developed for major project components (e.g. pipe, manholes, service connections, gravel, pavement, etc.) and used to estimate the cost of the various diameter pipelines. The pipeline costs are based on depth of construction and include allowances for installation costs for labor and equipment associated with excavation, backfilling, and dewatering operations. Costs for relocating existing utilities other than resolving minor utility conflicts that occasionally arise during normal sewer construction were not included in this construction cost estimate but these are expected to be minimal for this project.

The unit costs used for this estimate are based on the June 2009 Engineering News Record (ENR) construction index of 8578 and are shown on Table 3-3. The ENR construction index is based on an average cost of key construction materials for 20 major U.S. cities. Use of this index allows for escalation of project costs in the future by comparing the current cost index to the index at the time of proposed construction. The costs in Table 3-3 were used for comparing the estimated present day construction cost of the various alternatives that were evaluated for sewering the project area (See alternatives analysis in Section 4). Total project cost of selected alternatives include other cost such as contingencies, design, escalation, etc. and are discussed in Section 5 with development of project costs.

Insert Table 3-3 – February 2009 cost.