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Upgrade of Wastewater Sistema Central in Havana, Cuba

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Abstract: *Sistema Central is the system that collects and disposes of most of the domestic wastewater in the city of Havana, Cuba. This system is currently in a state of disrepair and is a major source of pollution to the coastal waters of Cuba. A team at the University of Miami (UM) developed preliminary-level engineering designs to upgrade the key components of Sistema Central, including: a new ocean outfall with a multi-port diffuser, design of preliminary treatment, the system curve for selection of new pumps at the Casablanca pump station, and a disinfection system to reduce bacteria levels in the wastewater discharge. The ocean outfall was designed such that all ambient water-quality standards in the vicinity of the new outfall will be met. Parameters considered in designing the outfall included the coastal bathymetry, wind and current velocities, water-quality standards in the coastal waters, and flow rate to be handled by the system. In order to meet the water-quality standards, preliminary treatment, chlorination, and disposal through a multi-port diffuser was determined to be the best option.*

Keywords: *Cuba, infrastructure, ocean outfall, Sistema Central, wastewater*

Notations and symbols

A	*Cross-sectional wetted area*
B_0	*Specific buoyancy flux*
B_o	*Specific buoyancy flux*
C	*Concentration of the contaminant in the ambient water*
C	*Empirical discharge coefficient*
$c(x,0)$	*Maximum concentration along the plume centerline*
C_{BDNF}	*Near-field dilution constant*
C_{sss}	*Empirical constant*
$d_{\rho a}/dz$	*Density gradient*
f	*Friction factor*
F	*Fetch*
g	*Gravitational acceleration*
H	*Discharge depth*
h_0	*Total head*
$H_{1/3}$	*Significant wave height*
h_{fd}	*Head loss in diffuser*
h_{fo}	*Head loss in outfall*
h_{ft}	*Head loss in the tunnel*
h_n	*Thickness of the plume*
k	*Decay coefficient*
L	*Length*
L_b	*Plume-crossflow length scale*
L_D	Length of the diffuser
L_M	Plume transition length scale
M_0	Specific momentum flux
N	Buoyancy frequency
Q_0	Port discharge
R	Hydraulic radius
S	Dilution
$T_{1/3}$	Significant wave period
T_{90}	90% decay time
U	Wind speed
u_a	Ambient current speed
V	Velocity through the openings of the bar screen
x	Coordinate in the direction of flow
y	Coordinate in the transverse direction
z_m	Maximum height of rise
z_n	Height
ε_0	Initial diffusion coefficient
ε_y	Transverse diffusion coefficient
ρ_0	Ambient density at the discharge location
ρ_a	Ambient density at a distance z above the discharge location
v	Approach velocity in the upstream channel

1. Introduction

Wastewater collection and treatment systems are important aspects of the human water cycle and are built to safely collect and dispose of wastewater produced by human activities. Features of a properly designed wastewater system include adequate capacity to handle all wastewater flows from the service area, minimal impact on human activities nearby, and minimal environmental impact. As shown in Figure 1, the Sistema Central wastewater system has an 1130-km sewer network that handles the domestic wastewater collected from the following municipalities in Havana:

Habana Vieja, Centro Habana, Diez de Octubre, Cerro, Plaza de la Revolución and some districts in Arroyo Naranjo, Playa and San Miguel de Padrón (Alonso Hernández and Mon, 1996).

This system was originally designed for a population of 600,000 but is now serving over 945,000 people and currently has an average flow rate of approximately 2 m^3/s (45.6 mgd). As shown in Figure 2, wastewater is collected through two major sewer lines: Colector Norte and Colector Sur, which transport the wastewater to Muelle de Caballería for preliminary treatment. The existing system provides limited pre-treatment that includes 50-mm bar screens and settling of coarse material in a grit chamber. Both the bar screens and grit chamber have not been repaired since 1991; there is no readily available information on how well these components are operating.

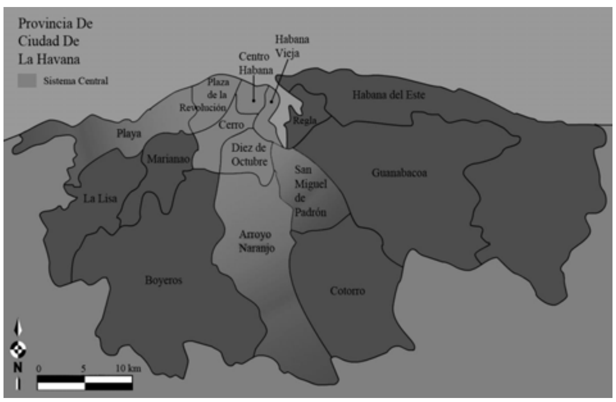

Figure 1. Map showing the municipalities in Havana served by Sistema Central

Following this preliminary treatment at Muelle de Caballería, the wastewater flows by gravity through an inverted siphon across the Bay of Havana to the Casablanca pump station from where it is pumped through a 1.5-km tunnel to the existing outfall at Playa del Chivo. This 1.5-km tunnel consists of a 1361 m pipe buried under the ridge of La Cabaña and a 114-m pipe constructed in an open trench at Playa del Chivo. The existing outfall discharges from a single port at a depth of 10.7 m through a cast iron pipe (1.5 m in diameter and 0.13 m in thickness) that extends 147 m offshore (Egues and González, 1997). The length and discharge depth of the existing outfall result in a dilution of 3:1 which is inadequate to meet the ambient water-quality standards of the surrounding coastal waters (Hernández, Cabañas, and Chabalina, 1997). In addition, the existing outfall pipe has multiple leaks, causing much of the wastewater to be discharged in shallow water before reaching the discharge port.

An upgrade to the current system is essential to meet the needs of the population in the service area and to meet the ambient water-quality standards in the coastal waters where the wastewater is discharged. The proposed system upgrade takes into account service flows at the beginning and end of the design period, and wave forces on the outfall structure, in order to produce an outfall design that will meet all ambient water-quality standards and handle design flows throughout its 30-year design period.

Alternatives such as maintaining the existing outfall, prolongation of the existing outfall with system optimization, construction of a new outfall with diffuser and construction of a new outfall that discharges at a deeper depth were set forth and compared in a conceptual design of an ocean outfall at Playa del Chivo (Hernández, Cabañas, and Chabalina, 1997). Parameters such as depth of wastewater discharge, flow rate, number of ports in diffuser were varied and an initial dilution of wastewater achieved by each alternative was calculated as a measurement of efficacy. Constraints such as project cost and construction difficulty were also taken into account when each alternative was evaluated for feasibility. Based on this existing methodology, five alternatives were considered in this paper prior to selecting the preferred option for Sistema Central.

The first alternative is to construct an outfall that discharges at a sufficient distance from shore where there are no statutory water-quality standards to be met. This alternative would have the advantage that chlorination of the wastewater would not be necessary, thereby avoiding the high operational and maintenance costs associated with using chlorine. However, this alternative would result in an extremely higher construction cost due to the depth of the outfall because expensive diving operations will be necessary (Roberts, 2010). To avoid having to meet water quality standards, the outfall would have to be at least 2.5 km offshore with a corresponding depth of approximately 213 m. Increased costs in this case would be due to an increase in pipe material and ballasts needed, and also installation and maintenance at the deeper depths. Between 740 m and 865 m offshore, the slope of the ocean floor is approximately 80% (U.S. Navy, 1964). Constructing an outfall to this distance would be practically infeasible due to the steepness of the ocean floor.

The second alternative is to construct an outfall at a depth of 45 m or less with no diffuser. This alternative was ruled out because the dilutions attained were calculated to be insufficient to meet coastal water-quality standards (Caribbean Environment Programme, 2009). Even though there exist outfalls in Florida, U.S., such as those for the Broward/North and Hollywood wastewater systems, which discharge at depths less than 45 m with only one port, both outfalls discharge at distances that are more than 2000 m offshore, which would not be economically practical for Havana (Koopman and Heaney, 2006).

The third alternative is not to design a new outfall, but to simply keep providing maintenance to the existing outfall. This alternative was determined to be impractical since the existing outfall is not able to provide sufficient dilution to meet the ambient water-quality standards. In addition, the existing outfall is approximately 100 years old and in such a state of disrepair that maintenance is insufficient and an upgrade is inevitable.

The fourth alternative is to design an upgrade to the existing outfall and supporting systems that would meet all ambient water-quality standards and perform adequately for at least the next 30 years. This alternative has the advantage in that it uses most of the existing infrastructure while replacing the most critical part of the system. A similar situation occurred in Puerto Rico where a gravity run 1500-m outfall was only sufficient to provide primary treatment before discharging into the shallow waters of the Bay of Ponce (Lorenzo, 2001). After the passage of the Federal Clean Water Act in which the U.S. Government required that all publicly owned treatment works must reach a minimum of secondary treatment to protect public waters, the outfall was upgraded by increasing the depth, which required the addition of a pump station, as well as a 45 m long diffusor to the end of the outfall. The upgraded outfall was able to successfully meet the required ambient-water quality standards and saved the area the costs of installing new secondary treatment systems.

The fifth alternative is to abandon ocean-outfall disposal, design new primary, secondary, and tertiary treatment systems, and use the treated (reclaimed) water as a freshwater resource for irrigation or industrial cooling. A major disadvantage to this alternative would be the amount of completely new and costly infrastructure necessary to successfully implement this system because a brand new design, pilot studies, and an extensive construction project from the ground level would have to occur. Even though many outfalls are becoming abandoned in the United States (Koopman and Heaney, 2006), there are many outfalls still present in the Caribbean such as the previously mentioned Ponce outfall in Puerto Rico (Lorenzo, 2001), as well as some others in Barbados and Jamaica (Roberts, 2010). In addition to the aforementioned disadvantage, there is not enough information on the interest and desire for such a dramatic change in wastewater treatment. The government would have to decide if this is the proper approach for their wastewater treatment, and if it would be in the best public interest. It was not possible to find in the available Cuban literature any indication of an inclination toward such an approach, and even less any necessary information such as right-of-way availability for new infrastructure. Based on the foregoing considerations, the fourth alternative of using an upgraded outfall along with improved supporting systems that will meet all water-quality standards was selected.

This paper describes the design of a new ocean outfall to replace the existing outfall servicing Sistema Central, and the supporting pump-station performance specifications and chlorination requirements. Site-specific guidelines for construction of the outfall and an opinion of probable cost of the proposed system are also included.

2. Design Parameters and Constraints

Parameters that are relevant to the design of the Playa del Chivo outfall include bathymetry of the coastal area surrounding the outfall, current speeds and directions, wind speeds and directions, the effluent discharge rate, and the ambient water-quality standards near the outfall. On the basis of these parameters the length and diameter of the new outfall, pipe material, diffuser characteristics, and treatment required for the wastewater were all determined.

2.1 Water Quality Classification Map

The classifications of the coastal waters of Cuba were identified from a map published by Caribbean Environment Programme (2009), and the classifications in the vicinity of Playa del Chivo are shown in Figure 2. Each classification has different water-quality standards, with the most stringent standards being in Class B waters, which are suitable for direct-contact activities such as swimming and bathing; the least stringent standards are in Class E waters, which are suitable for maritime port activities.

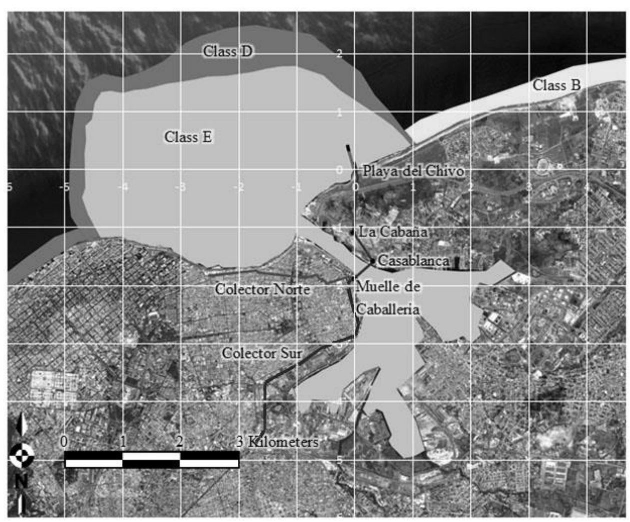

Figure 2. Map showing known wastewater system and classifications of receiving waters.

At Playa del Chivo, an outfall providing at least 100:1 dilution will meet all applicable water-quality standards with the exception of the fecal-coliform (FC) standard. The FC standard will be met by applying an appropriate level of chlorination to the effluent prior to discharge. The locations of both the existing outfall (Hernández, Cabañas, and Chabalina, 1997) and proposed outfall are shown in Figure 3, along with the location of the Casablanca pump station, and preliminary-treatment facility at Muelle de Caballería (see Figure 2). The proposed outfall is purposely located adjacent to the existing outfall so that it can be easily connected to the existing infrastructure.

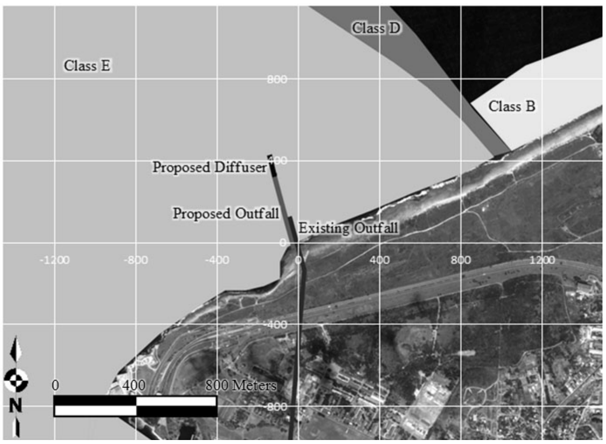

Figure 3. Receiving waters classification map showing locations of current and proposed outfalls

2.2 Bathymetry

The bathymetry in the vicinity of Playa del Chivo was obtained from a map prepared by the U.S. Navy and the Public Works of Cuba in 1964 (U.S. Navy, 1964) and linearly interpolated between the contours. The depth of water with distance from the shoreline along the outfall trajectory is shown in Figure 4.

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Figure 4. Depth profile of the proposed ocean outfall.

In light of limited data, the seabed extending between 300 to 600 m from the shoreline was assumed to be linear. However, it is likely that the sea floor is actually rocky and uneven. The slope of the ocean bottom was estimated as 13%, which is relatively steep compared to ocean outfalls found on the east coast of the United States.

2.3 Wind Speed and Currents

There is limited data on the coastal currents in the vicinity of Playa del Chivo. The most relevant data was that reported by Martinez et al. (1999) who deployed several current meters in the vicinity of the existing outfall. The results reported by Martinez et al. (1999) showed that the currents were mostly parallel to the coast, tidally dominated, and having minimal variation over depth. These data showed that 64% of the time the currents were towards the east with an average maximum velocity of 34 cm/s, and 32.5% of the time the currents were towards the west with an average velocity of 25 cm/s. A review of the wind data showed that an average onshore wind of 10 m/s could be expected in the windiest month of the year, and so the onshore/offshore surficial currents were estimated as 1% of this wind speed, yielding corresponding wind-induced currents of 10 cm/s. Using these available data, a current ellipse with alongshore major axes of 34 cm/s and 25 cm/s and onshore/offshore minor axes of 10 cm/s was developed. This current ellipse was used to estimate the average currents in all directions, which are essential parameters for estimating effluent dilution in the coastal waters of Playa del Chivo.

2.4 Flow Rate

The city of Havana has an estimated population of 2.4 million people of which 63% are connected to sewer systems, and 66% of those connected to sewer systems are connected to Sistema Central, making Sistema Central the largest sewer network in Havana (Egues and González, 1997). A conventional approach to projecting average flows of domestic wastewater is to extrapolate past population trends in the service area, and then multiply the extrapolated population by an average per-capita wastewater generation rate that usually remains relatively constant during the projection period. In the case of Havana, this conventional approach is not entirely appropriate since the current population trend in Havana is negative (i.e., the population is declining) and it does not appear prudent to project a population that will be less than the current service population in 30 years. A second consideration in projecting wastewater flows in Sistema Central is that under current conditions the per-capita wastewater flow rate is estimated to be approximately 150 L/person/day (40 gallons/person/day), while the future per-capita flow rate is expected to be approximately 570 L/person/day (150 gallons/person/ day) when the hydraulic capacity of Sistema Central is fully realized. Based on considerations of both the current declining population in Havana and the extent of the capital improvements necessary to restore the hydraulic capacity of Sistema Central, the design average flow rate at the end of the design period was taken to be the current population multiplied by the per capita wastewater generation rate corresponding to hydraulically adequate Sistema Central. This calculation yields a design flow rate of 6.68 m^3/s (152 mgd), which is 334% higher than the current average flow rate of 2 m^3/s (46 mgd).

3.Outfall Design

The main design objective is an outfall with a diffuser that is able to achieve a minimum dilution of 100:1, which is equivalent (disinfection issue aside for now) to secondary treatment without diffusers (Salas, 2000).

3.1 Pipe Selection

Choosing a durable pipe material is important in outfall design in order to achieve a longer design life with less need for maintenance. The pipe material selected for the outfall is high-density polyethylene (HDPE). HDPE is an excellent option for the outfall material because of its strength, quality, and flexibility. This pipe material is also easy to install because it floats, and has a relatively light weight that entails low transportation costs.

3.2 Hydraulic Design

The major hydraulic constraints of the design and their intent are listed as follows (Chin, 2013; Grace, 2009; Hernández, Cabañas, and Chabalina, 1997):

1. The bottom of the diffuser should be located at a water depth greater than 15 m to meet the conventional standards for Latin America and the Caribbean (Salas, 2000), and the diffuser depth should be less than 45 m below sea level to avoid complex and dangerous diving techniques.
2. The velocity in the outfall pipe should be greater than 0.6 m/s and less than 3 m/s to ensure that the critical velocity is achieved to prevent the deposition of suspended solids while avoiding excessive friction losses from high velocities.
3. The ratio of the total port area downstream to any pipe section should not exceed 0.7 and should ideally be in the range of 0.3-0.7 for the diffuser to flow full.
4. The port Froude number should be greater than 1 to maintain sufficient velocity to prevent seawater intrusion into the diffuser.

The design average flow rate was taken as 6.68 m^3/s based on the previously described flow projections. The diameter of the outfall pipe was taken as 1.6 m to achieve a sufficient scouring velocity to prevent sedimentation in the pipe. The port spacing between each port was taken as one third of the average depth between the first and the last port. This is the most efficient spacing because it prevents adjacent plumes from merging as they rise to the surface, as merging would reduce the amount of dilution. The length of the diffuser was calculated by multiplying the required number of ports by the port spacing.

In accordance with field measurements reported in the literature (Martinez et al, c. 1999), the ocean environment in which the effluent is discharged is approximately unstratified. One study reported a slight stratification in three small coastal areas during the rainy season, and horizontal variation in the temperature and salinity was very moderate, showing a weak gradient perpendicular to the coast (González-Sansón and Aguilar-Betancourt, 2005).

The conventional empirical formula for calculating the near-field dilution for a horizontal discharge from a diffuser port in unstratified ambient seawater is (Lee and Neville-Jones, 1987)

$$S = C_{BDNF} \frac{u_a L_b^2}{Q_0} \left(\frac{H}{L_b}\right)^{\frac{5}{3}} \quad (1)$$

where S is the dilution, C_{BDNF} is the near-field dilution constant, u_a is the ambient current speed, Q_0 is the port discharge, L_b plume-crossflow length scale, and H is the discharge depth. Equation (1) is valid when $H/L_b << 1$, and in the present case $L_b = 380$ m and $H/L_b < 0.12$ for all ports. Hence, utilization of Equation (1) to estimate the dilution of port discharges is validated. The performance characteristics of the final diffuser design are summarized in Table 1.

As shown in Table 1, under the design flow of 6.68 m3/s, the dilution across all ports is in the range of 65-158, and the average dilution across all twelve ports is 107, which fulfills the design objective of 100:1 dilution provided by the diffuser. The area ratios, flow velocities and Froude numbers at each port all satisfy the aforementioned design constraints. The first port of the diffuser is at a depth of 29 m and the last port is at a depth of 44.2 m. The cross-sectional diameters of the outfall pipeline were chosen according to commercially available HDPE pipe sizes, and the diameters of the diffuser sections range from 1.6 m to 0.5 m in order to maintain an adequate effluent velocity within the diffuser and so that the diffuser flows full. The total lengths of the outfall and diffuser are 340 m and 120 m, respectively, with port spacings of 10.95 m.

The average of near-field dilution in as a function of flow rate in an unstratified environment is shown in Figure 5. It is apparent from Figure 5 that under the existing flow conditions (2.01 m3/s), the outfall will achieve an initial dilution of 400:1, which is significantly higher than the dilution achieved at the design flow (6.68 m3/s). As the flow rate increases to the design flow, the initial dilution achieved decreases from 400:1 to 107:1.

Table 1. Performance Characteristics of the Diffuser

Port	Port Diameter (m)	Depth (m)	Flow (m^3/s)	Dilution	Area Ratio	Pipe Diameter (m)	Velocity (m/s)	Froude Number
1	0.3	29	0.58	65	0.42	1.6	3.32	27.91
2	0.3	30.4	0.58	70	0.39	1.6	3.05	29.11
3	0.3	31.8	0.58	75	0.35	1.6	2.77	29.22
4	0.3	33.2	0.58	81	0.32	1.6	2.49	29.32
5	0.3	34.5	0.55	93	0.5	1.2	3.94	27.64
6	0.3	35.9	0.56	100	0.44	1.2	3.45	27.98
7	0.3	37.3	0.56	106	0.38	1.2	2.95	28.28
8	0.3	38.7	0.57	113	0.31	1.2	2.46	28.55
9	0.3	40.1	0.52	133	0.56	0.8	4.43	25.93
10	0.3	41.5	0.54	137	0.42	0.8	3.32	26.97
11	0.3	42.8	0.52	151	0.45	0.63	3.57	26.24
12	0.3	44.2	0.53	158	0.36	0.5	2.84	26.61

The effects of stratification on the near-field dilution of the effluent were determined by calculating the buoyancy frequency, N, and subsequently determining the minimum near-field dilution in a stratified environment. The buoyancy frequency, N, is defined as

$$N = \sqrt{-\frac{g}{\rho_0}\frac{d\rho_a}{dz}} \tag{2}$$

where ρ_0 is the ambient density at the discharge location ($z = 0$), and ρ_a is the ambient density at a distance z above the discharge location. The density gradient, $d\rho_a/dz$, of a typical surface layer in the Atlantic Ocean is 9.667 x 10^{-3} kg/m^4 (Puyate and Rim-Rukeh, 2008). The minimum near-field dilution, $S_{\text{stratified}}$, under the influence of a stratified environment, is calculated as

$$S_{\text{stratified}} = \frac{C_{sss}\, B_0^{\frac{3}{4}}}{Q_0 N^{\frac{5}{4}}} \tag{3}$$

Where C_{sss} is an empirical constant, B_o is the specific buoyancy flux, Q_o is the flow rate and N is the buoyancy frequency. Using the buoyancy flux from each project, the resulting average dilution across twelve ports as a function of total flow rate was plotted in Figure 5 in comparison with the average near-field dilution in an unstratified environment.

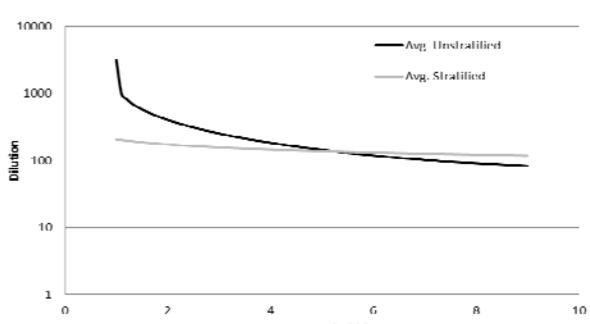


Figure 5. Average near-field effluent dilution in an unstratified environment and average near-field dilution in a stratified environment as a function of flow rate on a semi-log scale.

At the design flow rate (6.68 m^3/s), the average near-field dilution in a stratified environment was calculated to be 128:1 compared to the average dilution of 107:1 in an unstratified environment. As shown in Figure 5, as the flow rate exceeds 5.3 m^3/s, the average near-field stratified dilution appears to be higher than the average unstratified dilution because the equations used to calculate dilution in both stratified and unstratified environment assume pure plumes, but as the flow rate increases from 2 m^3/s to 6.68 m^3/s, the plume transition length scale, L_M, defined as

$$L_M = \frac{M_0^{\frac{3}{4}}}{B_0^{\frac{1}{2}}} \tag{4}$$

where M_0 is the specific momentum flux and B_0 is the specific buoyancy flux, accounts for up to 22% of the average depth of the diffusers. The plume transition length scale L_M measures the distance over which initial momentum is important. Therefore, not accounting for the influence of initial momentum as L_M increases can result in a higher dilution in the stratified environment.

The key geometric parameters of the plume: the thickness of the plume, h_n, the maximum height of rise, z_m, of the plume within the near field region, and the height, z_n, from the discharge level to the location of minimum dilution in the near field were calculated using the formulae by Daviero and Roberts (2006) and the results were summarized in Table 2. The thickness of the plume in the unstratified environment is taken as 30% of the total depth of diffuser and is also given in Table 2 as a reference.

Table 2. Key Geometric Parameters of the Plume in a Stratified Environment at the Existing and Design

	Q	h_n	z_m	z_n
Stratified	2.01 m^3/s	23.9 m	52.3 m	40.3 m
	6.68 m^3/s	32.3 m	70.7 m	54.5 m
Unstratified	/	13.3 m	/	/

The depths of the twelve ports in the designed diffuser range from 29 m to 44 m. Because both zm shown in Table 1 are greater than the 44 m, it was concluded that the plume is likely to surface in a stratified environment with a density gradient of 9.667 x 10-3 kg/m4. Therefore, stratification in the near-field is not likely to have a significant influence on near-field dilution at the design flow rate. At the existing flow rate (2.01 m3/s), the average near-field dilution in a stratified environment is 173:1 compared to 402:1 in an unstratified environment, which indicates that stratification can significantly lower the near-field dilution.

Far-field mixing occurs after initial plume dilution, when the buoyancy fluxes of the effluent plume are overwhelmed by ocean currents. (Chin, 2013). The effluent concentration at each point in the far-field was determined by using the Brooks (1960) solution to the steady-state advection-diffusion equation,

$$u_a \frac{\partial c}{\partial x} = \frac{\partial}{\partial y}\left(\varepsilon_y \frac{\partial c}{\partial y}\right) \tag{5}$$

where c is the concentration of the contaminant in the ambient water, x is the coordinate in the direction of flow, y is the coordinate in the transverse direction, and ε_y is the transverse diffusion coefficient. Brooks (1960) assumed that the transverse diffusion coefficient, ε_y, increases with the size of the plume in accordance with the four thirds law originally proposed by Richardson (1926) and supported by Okubo (1971). The maximum concentration along the plume centerline, $c(x,0)$, is given by

$$c\,(x,0) = c_0 e^{\frac{-kx}{u_a}} \tag{6}$$

where k is the decay coefficient, which is frequently expressed in terms of the 90% decay time, T_{90}, and β is defined by

$$\beta = \frac{12\varepsilon_0}{u_a L_D} \tag{7}$$

where ε_0 is the initial diffusion coefficient and L_D is the length of the diffuser.

Two separate mixing zones were delineated by calculating the boundaries beyond which the fecal coliform levels were less than 200 CFU/100 mL (CFU = Colony Forming Units) and 1000 CFU/100 mL to meet the ambient water-quality standards. As shown in Figure 6, to meet the ambient water-quality standard of Class B (c = 200 CFU/100 mL) with an adequate safety interval, the boundary of the 200 CFU/100 mL mixing zone is designed to be within 1000 m from the diffuser. As shown in Figure 7, to meet the ambient water-quality standards of Class D and E (c = 1000 CFU/100 mL), the boundary of the 1000 CFU/100 mL mixing zone should be within 50 m of the diffuser.

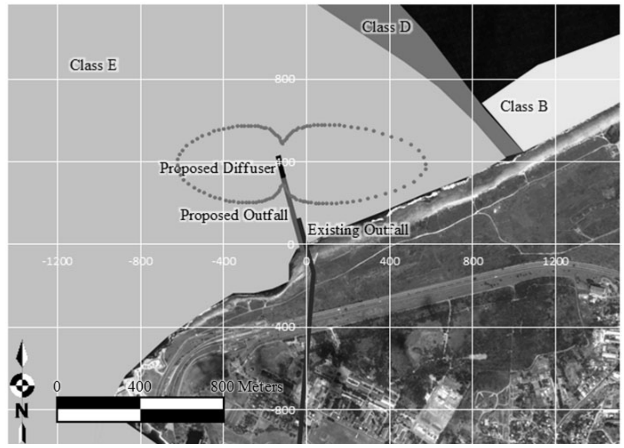


Figure 6. Boundary of mixing zone: c = 200 CFU/100 mL

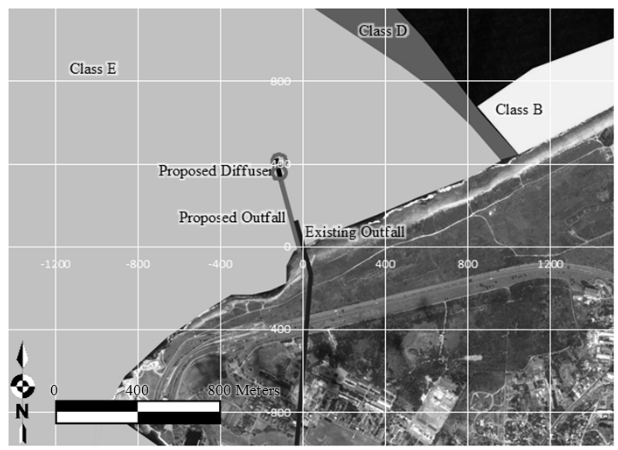


Figure 7. Boundary of mixing zone: c = 1000 CFU/100 mL.

3.3 Pump Selection

Prior to reaching the outfall at Playa del Chivo and being discharged, the wastewater is transported from Casablanca pump station through a 1.4 km underground tunnel below La Cabaña. The total head above sea level required for the diffuser to discharge the design flow is given by

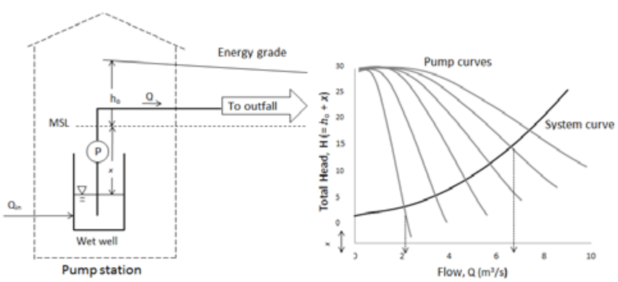
$$h_0 = h_{ft} + h_{fo} + h_{fd} \tag{8}$$

where the total head h_0 is the sum of the head loss in the tunnel h_{ft}, outfall h_{fo} and diffuser h_{fd}. The head-loss components in Equation (5) were estimated using the relation

$$h_0 = Q_0^2\left[f_t\left(\frac{L_t}{4R_t}\right)\left(\frac{1}{2gA_t^2}\right) + f_o\left(\frac{L_o}{4R_o}\right)\left(\frac{1}{2gA_o^2}\right) + \sum_{i=1}^{11} f_i\left(\frac{L_i}{4R_i}\right)\left(\frac{\epsilon_i}{2gA_i^2}\right)\right] \tag{9}$$

Where Q_0 is the flow rate in the outfall, f represents the friction factor, L represents the length, A represents the cross-sectional wetted area, R represents the hydraulic radius, where the subscripts "t", "o", and "i" are used indicate that the parameters refer to the tunnel, outfall, and section of the diffuser, respectively, and ε_i is the fraction of the outfall flow between ports i and $i + 1$.

The total head above sea level (h_0) required for the existing flow (2.01 m^3/s) and design flow (6.68 m^3/s) were calculated to be 2.23 m and 14.65 m, respectively, and these operating points are indicated by arrows originating on the system curve in Figure 8(b). The final operating points will be determined by the performance curves of the selected pumps; however, further investigation needs to be done to finalize a design of the pump system. At the pump station, the elevation of the wastewater in the wet well below sea level (x), as illustrated in Figure 8(a), needs to be determined onsite to be able to calculate the total head needed by the pump. Once this information is obtained, the appropriate pumps can be selected based on the head needed for the projected flows.



(a) (b)

Figure 8. System curve required for Casablanca Pump Station.

3.4 Disinfection

Prior to the wastewater being discharged through the ocean outfall, disinfection is necessary to meet the FC water-quality requirements. Chlorination is the preferred method of disinfection. Chlorine is an excellent disinfectant of wastewater and, given the right dose, chlorine contact with the wastewater will achieve a high degree of removal of FC in the wastewater. To properly chlorinate the wastewater, a contact time of approximately 30 minutes is desirable. The detention time from Casablanca to the end of the outfall was calculated to be about 15 minutes. To attain a total contact time of 30 minutes, a 15-minute detention tank will need to be constructed at Casablanca. To achieve a detention time of 15 minutes at a design flow of 6.68 m^3/s, a 5313 m^3 volume tank would be needed. The chlorination dosage of 10 mg/L was determined by adjusting the percent removal of fecal coliform so that the boundary of mixing zone lies within the ambient water-quality standards. A 3-log reduction, which corresponds to 99.9% reduction of FC needs to be achieved in order to meet the ambient water-quality standards for Classes B, D and E.

In addition to chlorination, preliminary consideration was given to the possibility of using ultraviolet light, but this option was discarded early on because of the uncertainty of the effectiveness of this method in wastewater with high solids concentration, as is the case here. Also, potential toxicity effects that are sometimes associated with chlorine were not found to be of particular concern in this setting.

3.5 Preliminary Treatment

Prior to outfall discharge at Playa del Chivo and proposed chlorination at Casablanca, the wastewater undergoes preliminary treatment at Muelle de Caballería. Currently, the treatment includes the removal of solids through bar screening and settling of coarse materials in a grit chamber. Since the status of the existing preliminary treatment system is unknown, an initial design of a new preliminary treatment system was conducted.

The use of bar screens is an effective way of removing large solids from the wastewater. By removing the larger solids, essential components such as pumps, valves, and pipelines are protected from being damaged or clogged. There are a variety of options to routinely clean the bar screens since the bar screens will become clogged over time. Bar screens are either hand or mechanically cleaned. Hand cleaned bar screens are used for small to medium flows and are common in bypass channels during high flow periods. The sizes of hand cleaned bar screens are limited to allow for convenient cleaning. Usually, mechanically cleaned bar screens are preferred because they are less labor intensive.

Typical mechanically cleaned bar screens include chain-driven screens, reciprocating rake (climber) screens, catenary screens, and continuous belt screens. The continuous belt screen is continuous, self-cleaning and has no submerged sprocket. This screener is difficult to jam and maintenance can be done above the operating floor. Because of these ideal advantages, the preferred bar screen for this system is a continuous belt screen (Tchobanoglous, 2003).

When designing a bar screen, factors to consider include: location, approach velocity, clear openings between bars or mesh size, and head loss through screens. The head loss through the bar screens is a function of the approach velocity and the velocity through the bars as shown in equation (10) (Tchobanoglous, 2003):

$$h_L = \frac{1}{C}\frac{V^2 - v^2}{2g} \qquad (10)$$

where C is an empirical discharge coefficient to account for turbulence and eddy losses (0.7 for clean screen and 0.6 for clogged), V is the velocity through the openings of the bar screen (m/s), v is the approach velocity in the upstream channel (m/s), and g is the acceleration due to gravity (m/s^2). Table 3 shows typical design parameters for bar screens.

Table 3. Design Information for Manually and Mechanically Cleaned Bar Screens

Parameter	Unit	Cleaning method	
		Manual	Mechanical
Bar size:			
Width	mm	5-15	5-15
Depth	mm	25-38	25-38
Clear spacing between bars	mm	25-50	15-75
Slope from vertical	degree	30-45	0-30
Approach velocity:			
Maximum	m/s	0.3-0.6	0.6-1
Minimum	m/s		0.3-0.5
Allowable head loss	mm	150	150-600

Source: Adapted from Tchobanoglous (2003)

The next step in the preliminary treatment involves grit removal via grit chambers. Grit chambers are designed to remove sand, gravel, and other solids by settling. The purpose of this process is to protect mechanical equipment and reduce pipeline deposits. Three different types of grit chambers include horizontal flow, aerated, and vortex. Due to its ability to have consistent grit removal for different flows and its versatility, an aerated grit chamber is the preferred process (Tchobanoglous, 2003).

The first step of the design process is to establish the design flow, which is 6.68 m^3/s. The next step is to determine the chamber volume. In order to perform routine maintenance, two chambers are used. Utilizing an average detention time of 3 minutes, the volume for the design flow is 601 m^3 for each chamber. Next, the dimensions of each grit chamber are calculated using the width to depth ratio of 1.2:1. Using a depth of 3 m, the width is 3.6 m, and the length is 56 m. To determine the air supply requirement, an assumption of 0.3 m^3/min-m of length is adequate. Using this assumption, the air required for each grit chamber is 16.8 m^3/min and the total air requirement for both is 33.6 m^3/min. To calculate the volume of grit, a value of 0.05 m^3 grit/10^3 m^3 for wastewater is used, and the resulting volume of grit is 28.26 m^3/day (Tchobanoglous, 2003).

3.6 Force Analysis and Ballast Design

Since the outfall will be constructed using HDPE pipe, concrete ballasts must be installed in order to position the pipe on the bottom of the ocean and protect the pipe from oceanic forces. The ballasts must be large enough to hold the outfall in place, and the anchors must be properly spaced. If the ballasts are spaced too far apart, the bending force on the HDPE pipe could result in damage. If the ballasts are spaced too closely, the cost of the outfall could be much more than necessary.

To calculate the forces on the outfall, wind data from the National Data Buoy Center (Buoy 42002) was used. Buoy 42002 is a buoy that is located approximately in the direction of maximum fetch (distance of wind and water interaction) distance towards Playa del Chivo. Using the maximum wind speeds each year in the direction of Playa del Chivo, the wind speed distribution was best fit to a Dagum distribution using the Kolmogorov-Smirnov test. The design wind speed with a 50-year return period was calculated, resulting in a wind speed of 17.92 m/s. A fetch distance of 1550 km was used, corresponding to the longest unobstructed distance that allows waves to build up. From wind speed and fetch, the significant wave height $H_{1/3}$ and significant period $T_{1/3}$ were calculated using the empirical Wilson formulae (Roberts, 2010):

$$\frac{gH_{1/3}}{U^2} = 0.30\left\{1 - \left[1 + 0.004\left(\frac{gF}{U^2}\right)^{1/2}\right]^{-2}\right\} \quad (11)$$

$$\frac{gT_{1/3}}{2\pi U} = 1.37\left\{1 - \left[1 + 0.008\left(\frac{gF}{U^2}\right)^{1/3}\right]^{-5}\right\} \quad (12)$$

where U is the wind speed in m/s, F is the fetch in m, and g is gravitational acceleration in m/s^2. Applying equations (11) and (12), the 50-year design wave has significant wave height $H_{1/3}$ = 7.02 m and significant period $T_{1/3}$ = 11.32 s. These values were then used to determine the wave forces on the pipe. The maximum wave forces were calculated using the U.S. Army Corps of Engineers wave analysis programme, Automated Coastal Engineering System (ACES), which is part of the Coastal Engineering Design and Analysis System (CEDAS). Using the linear wave theory analysis in ACES and varying the wavelength ratio resulted in a combination of velocities and accelerations that produced the maximum horizontal and vertical wave forces.

After calculating the maximum wave forces, the concrete ballasts were sized by a total-force analysis on the pipeline. Force balance calculations revealed that the forces controlling the sizing of the ballasts were the vertical forces. The vertical forces were balanced by changing the volume per meter of the concrete ballasts, and ballast volume per meter was calculated at depths ranging from 10 m to 25 m. The concrete used in the calculations is high-density concrete with a density of 3840 kg/m^3. Preliminary cost analysis was performed to determine the depth to which the pipeline should be buried. The cost analysis calculated the trade-off between excavation cost and concrete ballast cost at different pipe burying depths. A pipe buried to a deeper depth would require more excavation cost but lower concrete costs. The result was that it would be possible to keep the pipe buried only to a depth of 10 m, corresponding approximately to the wave-breaking zone for the 50 year design wave. Using a safety factor of 1.5 to account for stress on the pipeline material from wave and tidal forces, it was determined that the pipeline should be buried for water depths less than 15 m.

After obtaining the volume per meter of the ballasts starting at 15 m depth, the ballast shape and dimensions were determined. Recommended spacing between ballasts using a HDPE pipe is no more than 30 m (City of Tacoma, 2012). A spacing of 25 m was used in the calculations. At this spacing increment and a pipe burial to a water depth of 15 m, 11 concrete ballasts are required. A trapezoidal ballast design is recommended as this ballast shape is widely considered to be the most stable (Roberts, 2010). The dimensions of the ballasts are shown in Table 4, and can be seen in Figure 9.

Table 4. Concrete Ballast Dimensions.

Concrete Ballast Dimensions	
Spacing (m)	25.00
V_{req} (m^3)	16.28
Height (m)	2.60
Base (m)	3.50
Top Width (m)	2.00
Thickness (m)	3.20
V_{actual} (m^3)	16.45

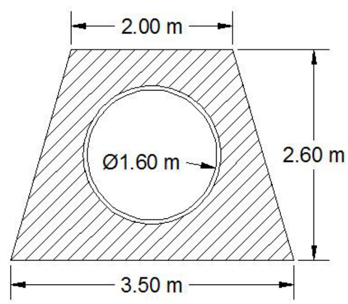


Figure 9. Cross section of a trapezoidal concrete ballast around the pipe.

4. Construction

Seasonal conditions are important in determining when to construct the outfall. The ideal time of year to construct the outfall is when the seasonal winds and seas are at its calmest. The wind speeds in Havana tend to be higher during the spring months, and the hurricane season in Cuba runs from June 1 to November 30. Taking these seasonal conditions into account, it is recommended that construction of the outfall begin at the end of the hurricane season to avoid potential hurricane force winds, as well as the higher winds in the spring.

The geological characteristics on the ocean floor influence the excavation that is required to lay a portion of the outfall beneath the ocean floor. The portion of the outfall closest to the coast is best designed to be below the ocean floor, which protects the pipe from wave and tidal forces and will also allow for a constant downward slope in the outfall pipe. The ocean floor off the coast of Havana is composed of (limestone) rock. There are a few ways of excavating through a rock surface. One way to excavate rock is through the use of explosives. This method is typical with large diameter pipes and requires a high level of experience. Since the proposed outfall is not very large, it is recommended that a hydraulic percussion drill be used. The advantage to this method is that it allows for more vertical trench walls, as well are a more accurate excavation.

The HDPE pipe can be transported to the site by a container in 12 m sections or can also be special-ordered in lengths up to 500 m. There are advantages and disadvantages to both methods. Shorter sections are easier to deal with as well as to transport; however, the welding of the pipe could potentially create leaks if done improperly. The long pipe would eliminate the need to weld sections together; however the shipment of a pipe this large would be infeasible. It is recommended to deliver the pipe in 12-m sections and have an experienced welder fuse the components together.

The installation of the pipe is facilitated by the fact that the pipe will initially float prior to being filled with water. The best installation method is to weld and fasten the ballasts onto the pipe on land. Once completed, the entire pipe can be towed out to sea. Again, this step would be best done during calm winds and calm seas. Once correctly positioned, the pipe can be filled with water and thereafter the it will sink to the bottom. The excavated area can then be backfilled, covering the pipe closest to shore.

5. Cost Analysis

A preliminary engineering-level opinion of probable cost was developed for upgrading the system. The results are summarized in Table 5. Major cost components include chlorination (Solomon, Casey, Mackne, and Lake, 1998) and the chlorination tank (Easton Suburban Water Authority, 2013). The majority of the other costs were calculated using information from Roberts (2010). Costs for pump upgrades were excluded because, as discussed earlier, onsite field data would need to be collected to determine the pumps used. Costs for preliminary treatment upgrades were excluded because the condition of the existing preliminary treatment system is unknown. Once this is known, it can be determined if the existing treatment system is adequate or if it requires maintenance or replacement.

6. Conclusion

Sistema Central is the largest wastewater collection network in the city of Havana and is currently providing inadequate treatment and disposal of the collected wastewater. The component of the system that is mostly responsible for this inadequacy is the ocean outfall that discharges the wastewater off the coast of Playa del

Table 5. Preliminary Opinion of Probable Cost of Upgrading the Wastewater System Excluding Pumps and Preliminary Treatment

Item	Unit	Quantity (Unit)	Unit Price (US $/unit)	Total	% of total
Pipe Supply	m	459	50	22,964	0.28
Excavation of earth and rock	m^3	731	589	430,624	5.25
High density concrete ballasts	m^3	181	500	90,475	1.10
Platform and auxiliary structure	m	60	4,710	282,618	3.45
Land platform		1	20,948	20,948	0.26
Civil works and outfall on land		1	141,057	141,057	1.72
Launching of outfall		1	301,132	301,132	3.67
Chlorination (10mg/L Dose)		1	3,000,000	3,000,000	36.59
Chlorination Tank		1	1,000,000	1,000,000	12.20
Total direct costs:				5,289,817	
Indirect costs at 30% of direct costs:				1,586,945	19.35
General expenses, profit and contingencies at 25% of direct costs:				1,322,454	16.13
Total:				8,199,217	100.00

Chivo. This existing ocean outfall consists of a single-port outfall (without a diffuser) that discharges wastewater at a depth of 10.7 m, providing insufficient effluent dilution to meet the water-quality standards of the surrounding coastal waters. In addition, the pipe has chronic leakage problems, causing the wastewater to be discharged at depths less than 10.7 m, thereby causing very high bacteria levels on the nearby coastline. There is no treatment provided to the wastewater aside from preliminary treatment consisting of bar screening and coarse sedimentation in grit chambers.

The intent of the study reported in this paper was to preliminarily design an upgrade to the system that would provide the equivalent of secondary treatment in terms of impact on receiving waters and meet coastal water-quality standards. The proposed outfall will have an average design flow of 6.68 m^3/s, which is three times the current average flow rate. Using state-of-the-art design methods and analyses, the updated outfall should have a length of 340 m with a 120-m multiport diffuser containing 12 ports. The HDPE outfall pipe should be buried to a depth of 15 m, and protected from wave forces with 11 concrete ballasts starting at 15 m depth with specifications given in Table 4. An effluent dilution of 107:1 at the design flow rate will result in meeting ambient water-quality standards. Higher dilutions will be achieved between when the outfall is first installed and decrease over time to match the aforementioned 107:1 when the actual average flow rate becomes equal to the design flow rate. Chlorine treatment will be required to meet fecal coliform standards using a dosage of approximately 10 mg/L, and a detention tank with volume of approximately 5300 m^3 will be required to achieve sufficient detention time. Pump station upgrades will be necessary, and the system curve that is the basis for selecting the appropriate pumps was developed as part of this study. If the existing preliminary treatment is insufficient to meet the needs of the new system, upgrade or replacement of the bar screens and grit chambers should be implemented as addressed in the preliminary treatment section.

If the designs presented in this paper are implemented, the Sistema Central with its new ocean outfall should provide an adequate level of service to the citizens of Havana for at least 30 years after it is constructed. The outfall will be able to handle maximum flows during the design period and will meet all water quality standards of surrounding waters.

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