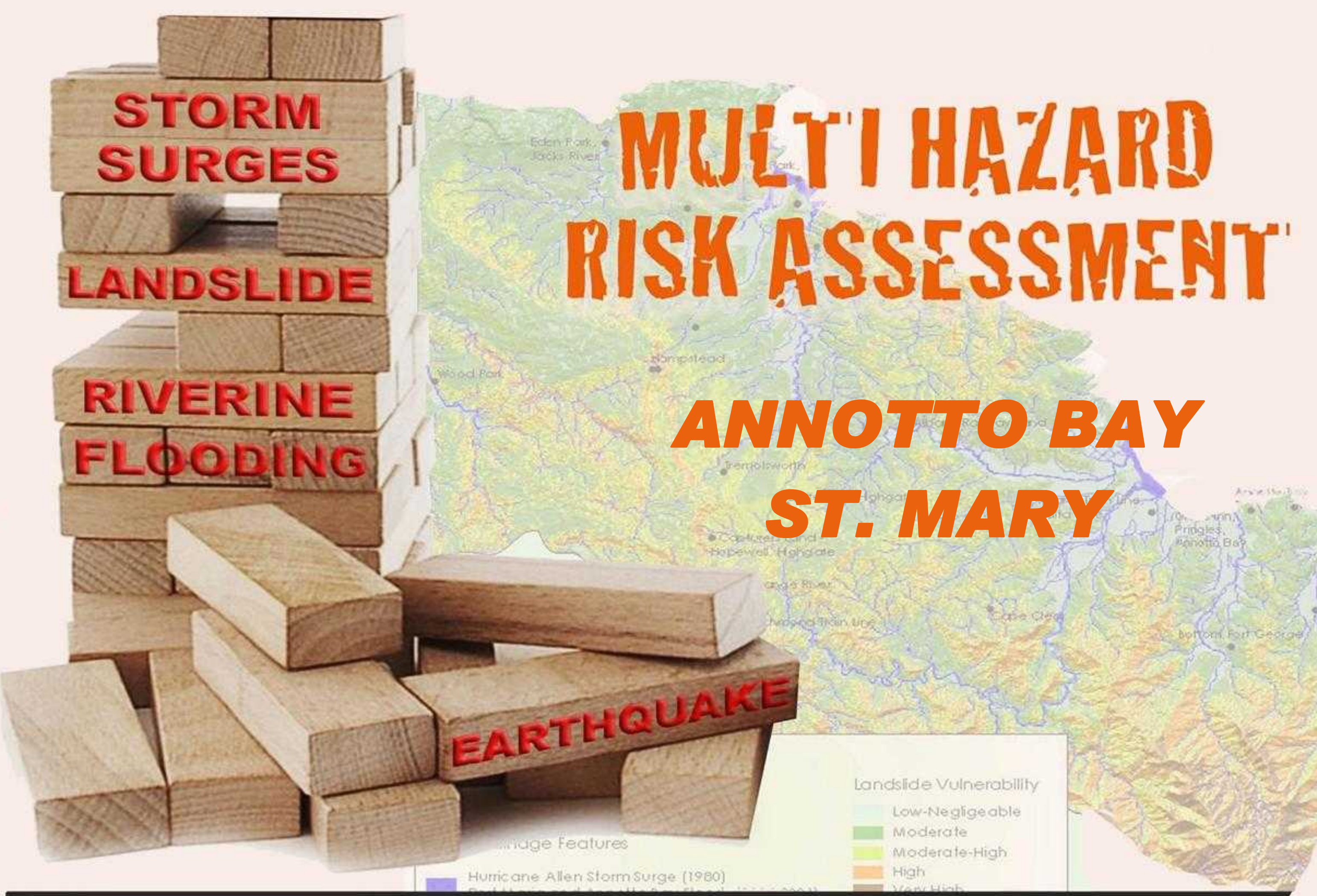
MULTI HAZARD RISK ASSESSMENT

ANNOTTO BAY ST. MARY



MITIGATION, PLANNING AND RSEARCH DIVISION

Funded by the Environmental Foundation of Jamaica

MULTI-HAZARD RISK ASSESSMENT

ANNOTTO BAY, ST. MARY

2013

Prepared by

Mitigation, Planning and Research Division

Office of Disaster Preparedness and Emergency Management

Jamaica

For

Annotto Bay Community Development and Environment Association

July 2013

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- Water Resources Authority
- St. Mary Parish Council
- St. Mary Banana Estate/Jamaica Producers
- Burrowes and Wallace
- Critical Facilities – Schools, Fire, Police, Health facilities

EXECUTIVE SUMMARY

Jamaica is vulnerable to hurricanes and associated storm surges and strong winds, flooding, earthquakes and landslides. The accumulated cost of natural hazards to Jamaica since 2004 is JMD$118 billion and 55 casualties resulting from the occurrence of seven major major events between 2004 and 2010. This vulnerability has transcend to the community level where Annotto Bay is one of the high risk communities in the country.

The low lying coastal community of Annotto Bay is traversed by four (4) major rivers- Annotto, Pencar, Motherford and Crooked Rivers. A fifth river, the Wagwater River forms the western most boundary of the community. Annotto Bay has witnessed over 30 major flood and 6 storm surge events in the last century and 2 decades, respectively. Moreover, Annotto Bay is located in close proximity to the most active seismic zone in Jamaica, the Blue Mountain Block. These hazards pose a serious threat to the population, critical facilities and infrastructure as well as the community's economy. The commitment of the Office of Disaster Preparedness and Emergency Management (ODPEM) to building disaster resilient communities assisted the Annotto Bay Health and Environment Association to develop a project proposal in response to Environmental Foundation of Jamaica (EFJ) call for proposals in 2011 for '*Institutional Strengthening, Programme and Project Implementation*" to support disaster risk reduction for vulnerable communities. The project was implemented by the ODPEM in collaboration with the CBO, EFJ and a number of government and private partners listed above in the acknowledgements.

This report provides analyses and assessment of four (4) natural hazards in Annotto Bay – storm surges, coastal erosion, riverine flood and earthquakes. Well-established scientific methodologies, tools and techniques have been used to assess the hazards and mapping accordingly. For storm surges, the hazard mapping presents inundation severity in terms of storm surge height and maximum extent of flooding inland with respect to 25-year, 50-year and 100-year return periods. It was found that maximum extent of flooding inland for each corresponding surge height was estimated from the shoreline at 312m, 378m and 407m inland for the 25, 50 and 100 year return period, respectively. Consequently, most of the developments from the coastline to just south of the highway, particularly the town centre are vulnerable to storm surges. The riverine flood hazard mapping is based the last major flood event associated with the passage of Tropical Storm Michelle in 2001. The areal extent of inundation covered approximately 436.0 hectares of land across Annotto Bay impacting 90% of the town. The coastal erosion mapping is for done 10 year return period storm event. For earthquakes, analysis is done based on the Rapid Visual Screening (RVS) which determines whether or not a building is seismically hazardous and requires detailed engineering assessment. The other methodology is the Nakamura H/V for determining local site effects which is a first step towards developing seismic microzonation map for Annotto Bay. Site effects associated with local geological conditions constitute an important part of any seismic hazard assessment.

The next component of this report is the vulnerability and risk assessment. Each hazard has specific impacts on particular sectors and four (4) types of elements at risk (buildings, population, roads and crops) were assessed. Of the total number of buildings in Annotto Bay which is estimated to be in the order of 1632, approximately 52% of these are exposed to earthquake hazard, 45% are exposed to riverine flood and 38% exposed to storm surges. As is expected the derived pattern of population exposure resembles that of building exposure and the findings indicate that the largest proportion of the population of 2708 persons are exposed to riverine flooding.

The risk analysis focuses on tangible losses associated with storm surge hazard scenarios both in terms of number of buildings as well as economic losses specifically for the critical facilities – schools, fire, station, police station, hospital and health centre in Annotto Bay. The expected average annualized loss is estimated at **JMD$ 13,007,517** for critical facilities only. The average annualized loss corresponds generally to the economic value that has to be paid annually in the long term to offset losses associated with future storm surge events.

Report Chapter Organization

The report is divided into six (6) chapters as follows:

Chapter 1 presents the introduction of the project. The chapter also describes project overview, objectives and expected outcomes.

Chapter 2 presents the geographic background, brief profile of natural hazards, and their respective trend in the community, characteristics of secondary data from various sources and various map information including administrative boundaries, population and topography.

Chapter 3 provides detailed understanding of hazard assessment, mapping and analysis of four (4) major hazards in Annotto Bay. The chapter elaborates the methodological approach for undertaking storm surge, coastal erosion and seismic hazard assessment as well as limitations where appropriate.

Chapter 4 discusses the exposure and vulnerability of the physical assets or elements at risk – population, housing, critical facilities, infrastructure and other buildings in the community to storm surge and riverine flood. For seismic vulnerability emphasis is placed on the seismic performance of buildings based on RVS methodology and zone (s) that are susceptible to liquefaction.

Chapter 5 presents the risk analysis based quantitative method that estimates the level of expected losses for a certain reference period, using the following equation: **Risk = H * V * A.** Losses associated with storm surge hazard scenarios is calculated for critical facilities only and expressed as average annualized losses.

Chapter 6 concludes with benefits of this risk assessment as well as some realistic recommendations for disaster risk reduction.

1.0 INTRODUCTION

Jamaica is highly vulnerable to a number of hazards namely, hurricanes and associated storm surge and wind, floods, landslides and earthquakes which have had adverse effects of people's livelihoods, physical infrastructure, the environment and on the economy. Between 2004 and 2010 the country experienced seven (7) major events with a price tag of approximately JD$108 billion and fifty five (55) casualties over the period.

At the same time, running parallel to these phenomena are emerging threats such as climate change, poor land use management, urbanization and environmental degradation are exacerbating vulnerabilities. Global climate change predictions suggest that the country could expect more intense hurricane and associated storm surge. Preliminary work also by the Climate Studies Group Mona at UWI suggests that intense rain events will increase across the island. Climatic variability is therefore adding a new dimension of current risk to the environment with over 400 of the island's 900 communities' ranked high and moderately high to natural hazards. A rise in sea level therefore could have devastating impact on coastal infrastructure and livelihoods as well as coastal towns and communities. The First National Communication indicated that the International Panel on Climate Change (IPCC) in 1990 estimated that the cost to protect Jamaica from one meter sea level rise would be $USD462 million.

Recognition of the adverse impacts of disasters on economic development, physical infrastructure, life and livelihoods, the Government of Jamaica (GOJ) has focused on the urgent need for disaster risk reduction. With the national mandate of comprehensive disaster management, the ODPEM has embarked on a number of initiatives to build the resilience of communities and by extension the country to be better able to withstand disasters and have enhanced ability to recover from associated impacts. As such, one of the strategic priorities of the organization is to conduct multi hazard risk assessment in urban coastal areas which will serve as a platform for informed decision-making. This priority of the ODPEM supports the United Nations Development Programme (UNDP) "Making Cities

Resilient" campaign as well as the Hyogo Framework for Action five priorities aimed at building the resilience of nations and communities to disasters. Moreover the "Making Cities Resilient" campaign has recognized that unless cities have a clear understanding of the risks they face planning for meaningful disaster risk reduction may be ineffective. Being party to both initiatives have underpinned the commitment of the ODPEM to comprehensive disaster management in Jamaica.

Other achievements, particularly in building resilience at the community level include the development of sixty five (65) community disaster risk management plans and community hazard maps with Annotto Bay being one (1) of the communities. Furthermore, twenty two (22) disaster risk reduction projects have been implemented in some of these communities.

1.1 Project Overview

This project is entitled "***Annotto Bay Disaster Reduction and Climate Change Adaptation Project***". This project addresses two (2) critical priorities identified in the Annotto Bay Community Disaster Plan:

i. reduction of flooding in the town;
ii. provision of data required for the implementation of mitigation and climate change adaptation solutions.

The Environmental Foundation of Jamaica (EFJ) had a call in 2011 for 'Institutional Strengthening, Programme and Project Implementation" to support disaster risk reduction for vulnerable communities. The Annotto Bay Health and Environment Association (ABHEA) submitted a request for funding to address the most pressing issues affecting the community. The EFJ found the proposal eligible and ABHEA was granted approval on July 15, 2011.

1.1.1 Project Objective

The main objectives of the assessment are to:

- conduct a multi hazard risk assessment of the four types of hazards - storm surge, flood, earthquake and coastal erosion affecting Annotto Bay.
- recommend appropriate cost effective mitigation measures that can be adopted and implemented to reduce the vulnerability of the community.

1.1.2 Expected Project Outcomes

The expected outcomes of this project seek to support initiatives and programmes of the community and the parish council to foster an integrated approach to Disaster Risk Reduction (DRR) and climate change adaptation in Annotto Bay. The four (4) main outcome areas envisioned are:

- Improved land use planning by incorporating multi-hazard risk assessment in the development approval process to prevent/control development in high risk areas or hazardous areas as well as incorporate disaster risk reduction and climate change impacts into the revised Development Order for St. Mary. The risk assessment will provide information to zone land use and prescribe restrictions on building type, use and density based on risk assessment.
- Provide geospatial data which will facilitate a better understanding of hazard, vulnerability and risk for improved decision making.
- Integrate risk assessment into existing Early Warning System (EWS) to improve response capability and preparedness efforts to ensure that the community can act in sufficient time and appropriately to reduce personal injury and loss of life.
- At the parish level, the assessment will inform mainstreaming DRR in various sectors.

2.0 Community Profile

2.1 Geographic Background

The low lying town of Annotto Bay is located on the north east coast of Jamaica with varying elevations of 1m to 3m above Mean Sea Level. The community is traversed by four (4) rivers along with their numerous tributaries – Annotto River, Pencar River and the Mother Ford River which has been converted to a culvert. The fourth, Crooked River lies to the east. The Wagwater and Dry Rivers form the western and eastern boundary of the community, respectively. The topography to the east, particularly in Iter Boreale boasts a higher elevation.

Figure 1 below shows a topographic map of Annotto Bay which highlights some of the main geographical features in the community. The project boundary as defined by Social Development Commission (SDC) is shown by the purple line which is bordered on the north by the Caribbean Sea, south by Foryland Pen, east by the Dry River and on the west by the Wagwater River.

The community sits primarily on two geological formations:

i. **Alluvium** - refers to material deposited by a river and represents an old floodplain or a part of an active one. This geological formation is characteristic of interbedded lenses of gravels, sands, silts and clay. In the case of Annotto Bay the community sits on an active floodplain of the aforementioned rivers.

ii. **Richmond Formation** – This formation is found on the steeper terrain of the community located in the east. The Richmond Formation is characteristic of very low slope stability and is highly prone to land slippage typically in the form of translational or rotational failures.

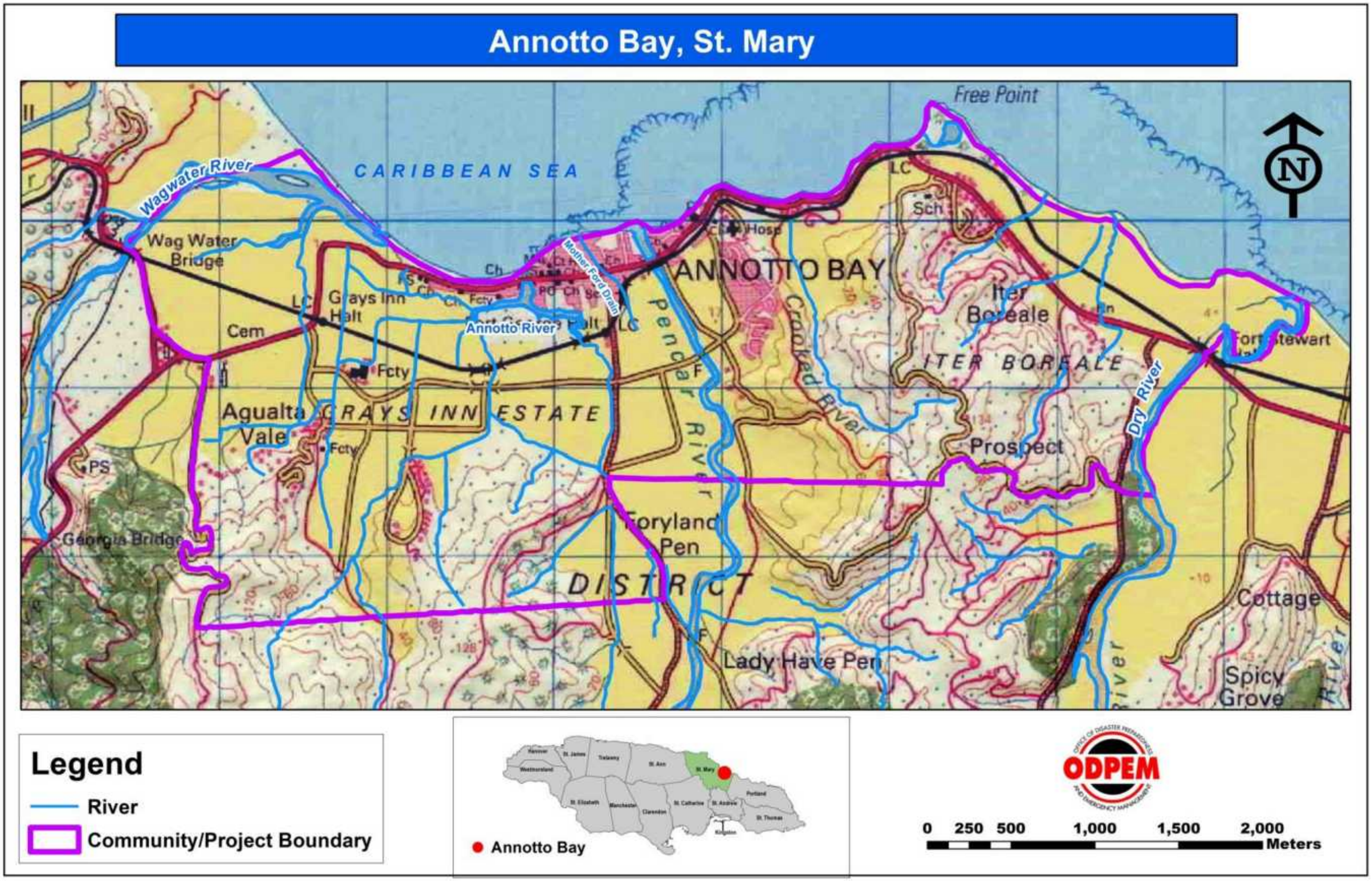


Figure 1: Topographic Map of Annotto Bay

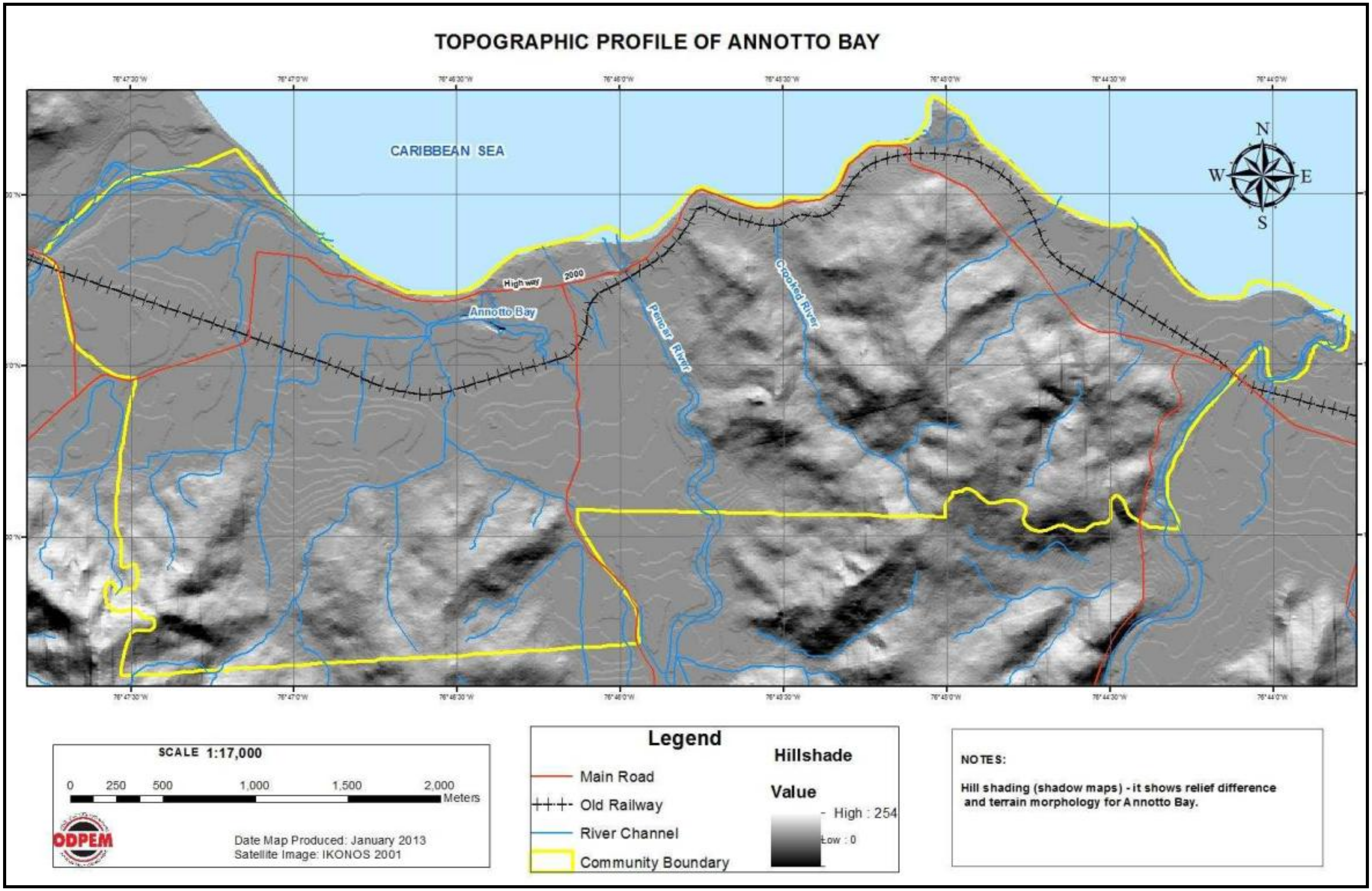


Figure 2: Topographic Profile of Annotto Bay

2.2 Natural Disaster Profile

Annotto Bay is vulnerable to a number of hazards by virtue of its topography and geographical location. The main hazards in the community are riverine flooding, hurricane induced storm surge and earthquakes. The profile shows that flood events are the most recurring with 35 such events occurring between 1901 and 2009 resulting in 9 deaths over the same period. These events have had devastating effects on physical infrastructure, critical facilities, economy and livelihoods. Annex 1 provides a detailed disaster profile for the community.

Table 1: Natural Disaster Profile of Annotto Bay

HAZARD	YEARS	NUMBER OF EVENTS	CASUALTIES	IMPACT
Riverine Flood	1901-2009	35	9 deaths 1 child missing	~JA$368 billion in damages for the parish
Storm surge	1980-2012	6	-	163 buildings destroyed
Earthquake	1812-1907	2	-	Caused tsunami effects
Tsunami	1812-1907	2		Max. wave height in 1901 – 9.1m

Source: Gleaner Archives and ODPEM Disaster Catalogue

2.2.1 Floods in Annotto Bay

Historical records show that flooding in the community is triggered mostly by heavy rainfall events and to a lesser extent by hurricanes. As aforementioned the causative factors for flooding relate to the location of the community being on an active floodplain of three (3) rivers coupled with the low lying topography of the area. Flood events of 1933, 1948, 1988, 1999 and 2001 are the worst to be recorded. The following is a synopsis of the impacts of the 1999 and 2001 flood events.

- *In March 1999 heavy rains over a two (2) day period caused extensive flooding of the Pencar River which resulted in the inundation of several homes and collapse of the Fort George Bridge isolating approximately 7,000 persons. The agricultural sector also suffered heavy losses.*

- *In October 2001, the Pencar River again overflowed its banks flooding an area of approximately 2km^2 including 90% of the town with flood depths of 2-4 feet. Over 300 households, businesses, schools and critical facilities including the fire station and hospital were affected The cost of damage to the housing sector in the Pencar-Buff Bay River Watershed was estimated at J$66M .*[1]

2.2.2 Storm Surge

The winds associated with a hurricane are the primary cause for storm surge. This phenomenon in Annotto Bay dates back as far as 1915 when an unnamed hurricane devastated the community killing four (4) persons, leaving more than 100 families homeless as well as demolished the two wharves, railway and other infrastructure in the town. The community was devastated again in 1980 during the passage of Hurricane Allen. The following account of the impacts of the hurricane is taken from Wilmot et al (1980):

- *Maximum surge height: 15 feet (4.6m)*
- *Maximum surge distance inland: 150 yards (137m)*
- *Structural Damage: 155 buildings were affected by seawater; serious damage was confined to waterfront areas. Many wooden houses were displayed from their foundation and the area of fire station was demolished*.
- *Coastal Alteration: There was extensive sand deposition and in some cases the shoreline retreated up to 30 feet.*
- *Additional Information: Residents reported similar sea levels during the 1951 hurricane (Hurricane Charley).*

[1] Geo Technics Limited. (2002). *Rapid Impact Assessment Project: Portland and St. Mary*. May 2002.

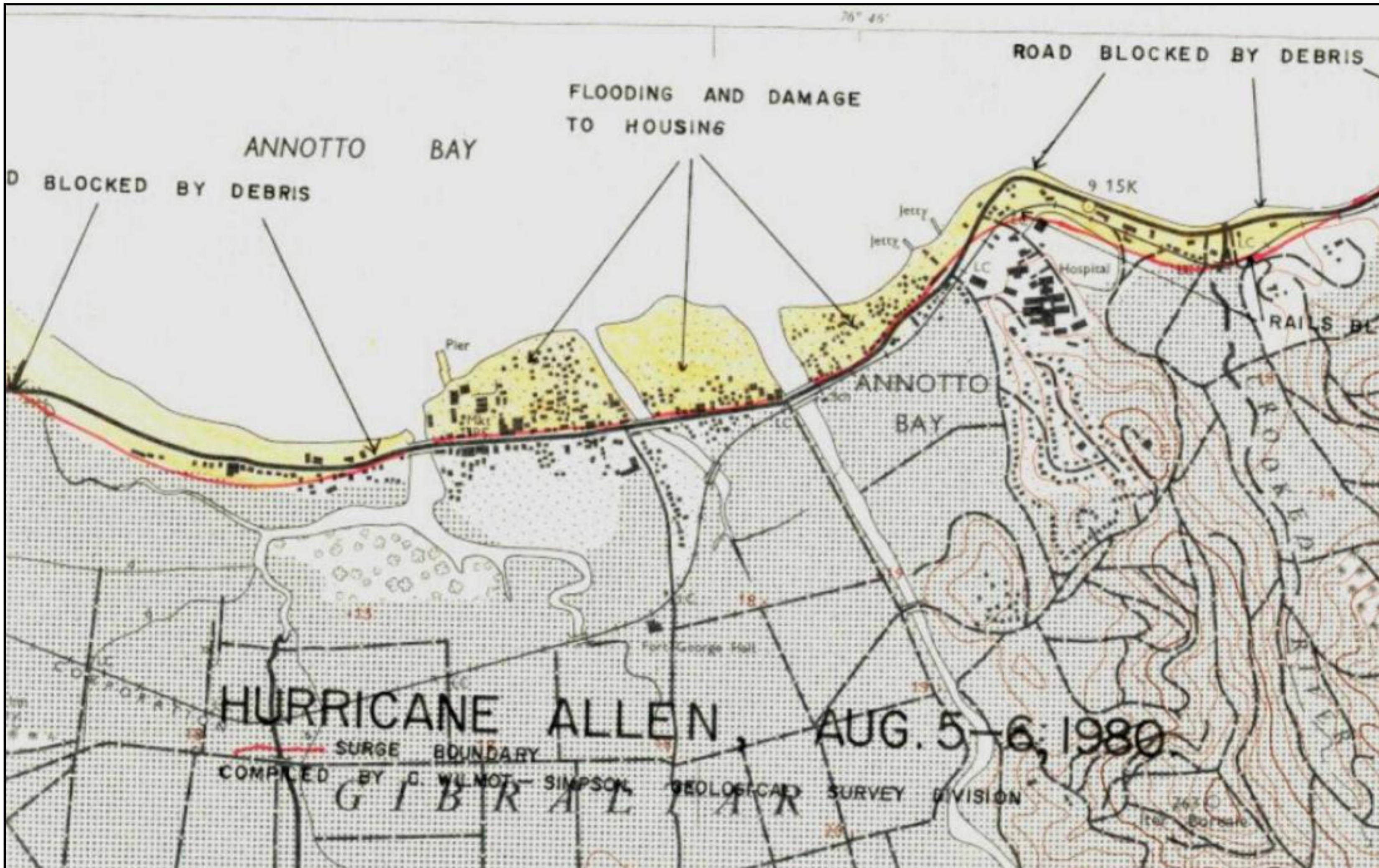


Figure 3: Hurricane Allen Storm Surge Boundary[2]

[2] Wilmot, C. et al. (1980). *Effects of Hurricane Allen Along the North Coast of Jamaica*. Geological Survey Division. September, 1980.

2.2.3 Earthquake Risk

History of Earthquake in Jamaica

The north-eastern section of the island has the highest frequency of earthquake activity. Although the precise epicentre of the major earthquakes of 1692 and 1907 are not known, based on intensity reports it is theorized that a likely location for the epicentres for these events would be in the north-eastern section of the island. There were also reports of tsunami in Annotto Bay after the 1907 which also adds credence to the theory that the epicentres of this event was within the north-eastern section of the island.

Based on the Earthquake Unit focal depth solutions it is clear that the hypocentre of earthquake events in this area are typically very shallow (~ 15 km) with typical fault offset in a left lateral motion. Focal mechanism solutions of recent earthquakes indicate a left lateral motion to be responsible for most fault dynamics.

Relatively high frequencies of earthquake occur in the Jamaica Seismic Network (JSN) sub-area (see figure 3); direct epicentres of event are rare in the town of Annotto Bay. However, even though there are no clear events in this location, the source area of most active events in the island is within 20 kilometres of this town. Thus very shallow earthquake and short distance of active faults coupled with poor construction practices, spells a high probability of serious earthquake damage for the study area.

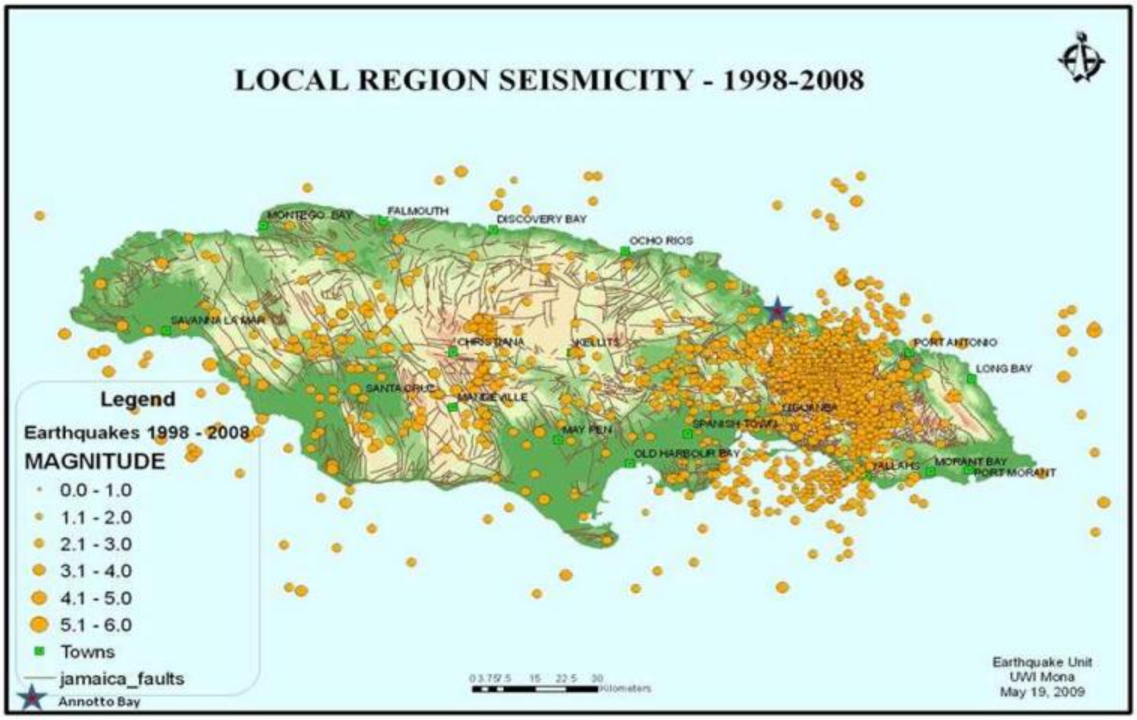
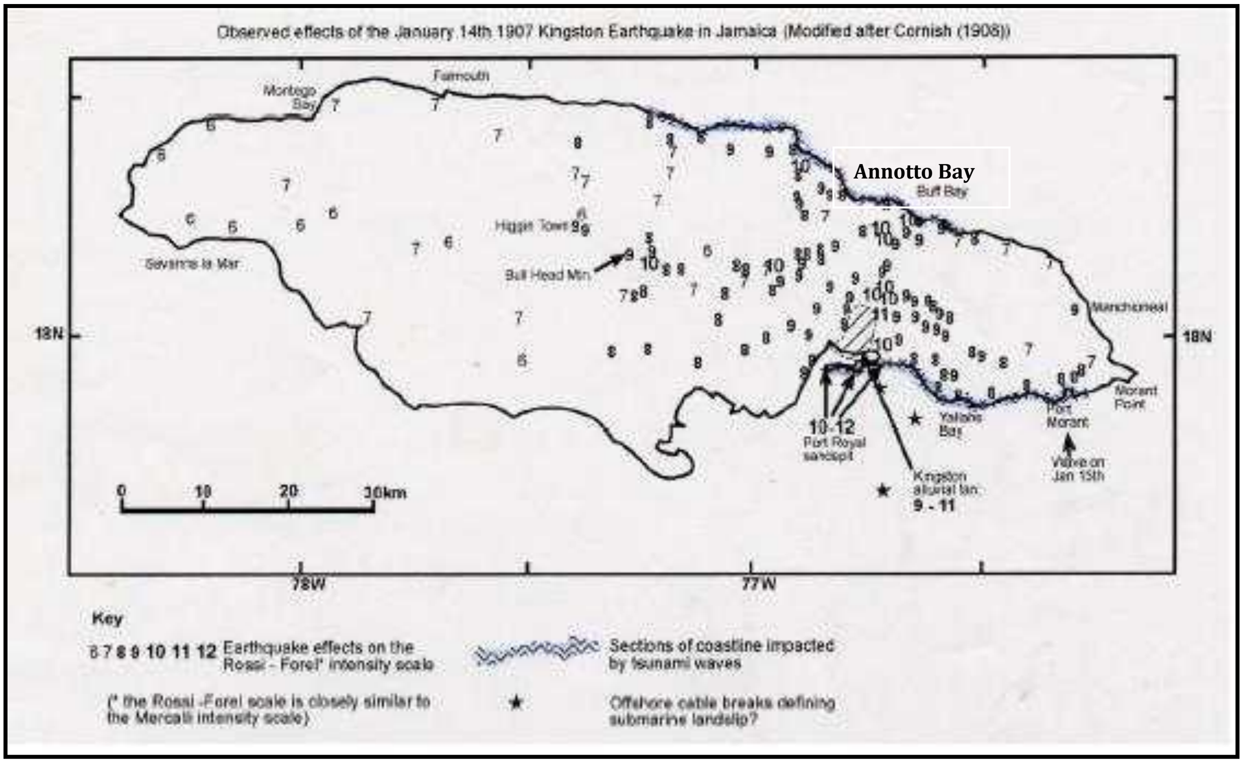


Figure 4: Epicentres of earthquakes in Jamaica during 1998-2008, highest frequency of events occurs in the north-eastern section of the island in close proximity to study area.

Historical records show that Annotto Bay directly suffered damage from two (2) major seismic events in 1812 and 1907. According to Lander et al 2002:

- ***November 11, 1812 -*** *the sea was much agitated following an earthquake. At Annotto Bay, Jamaica, anchorage ground sank causing a ship to lose its anchor and 90 fathoms (-180 m) of cable. This may be the description of the effects of a submarine landslide or of subsidence, or could be the description of a tsunami or the action of a seaquake.*
  `
- ***January 14, 1907 -*** *in Annotto Bay the sea receded 73 to 93 m, dropping 3 to 3.7m below normal sea level. The returning wave raised the water level 1.8 to 2.4m above normal, sweeping into the lower parts of the town destroying houses. On the higher land it came up 7.6 to 9.1m.*



wFigure 5: Observed Effects of the January 1907 Earthquake in Jamaica

Annotto Bay as shown on map experienced an intensity of 9 from the 1907 earthquake. According to the Rossi-Forel scale this means total or partial destruction of some buildings. The Rossi-Forel scale is a measure of intensity of shaking from an earthquake. This scale was replaced by the Mercalli intensity scale (Lander, 2002).

2.3 Soci-Economic Characteristics

2.3.1 Population Overview

According to the 2001 census, the population of Annotto Bay was estimated to be just over 5,000 people (5,422). Of the total, the number of females is slightly higher than that of males with figures of 2773 and 2649, respectively. The population is characterized as being youthful with 51.6% representing the 0-24 age cohort. Table 1 below represents the population structure for the community based on the 2001 census.

Table 2 : Population Structure for Annotto Bay

GENDER	0-14	15-64	65 AND OVER	TOTAL POPULATION 2001 CENSUS	POPULATION 2011 CENSUS
Male	934	1529	186	2649	
Female	894	1629	249	2772	
Total	1828	3158	435	**5422**	**6017**

Source: Statistical Institute of Jamaica, 2001

Since the 2001 Census, the population continued to grow, albeit not significant with an estimated population of 6,017 based on 2011 census. This represents a population increase of 595 persons or 11 per cent change.

Figure 6 below shows the population distribution by enumeration district. It can be seen that majority of the population is concentrated around the town centre with Enumeration District (ED) SE015 having the largest population of over 700 persons.

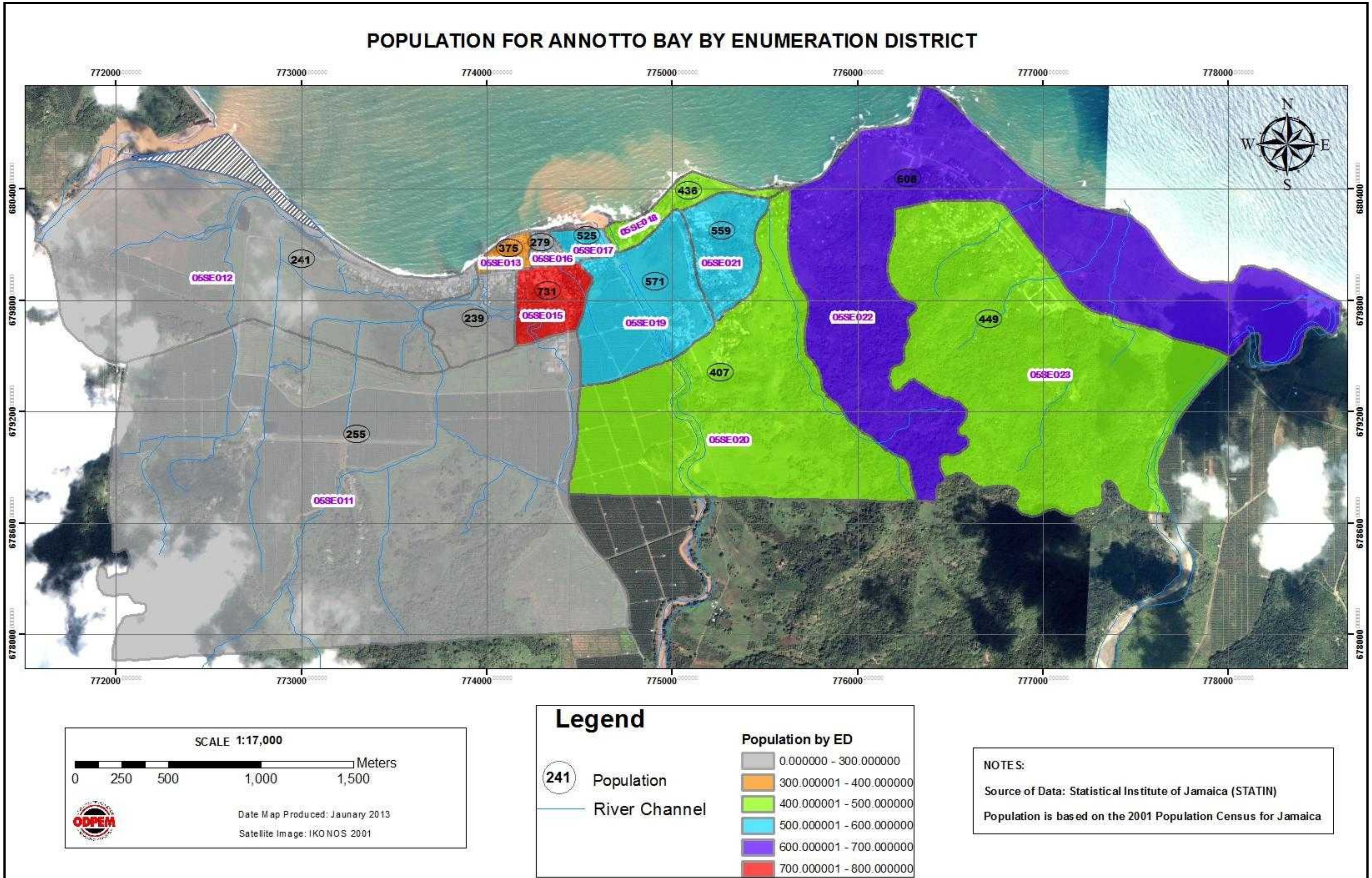


Figure 6: Population Distribution by Enumeration District

The 2001 Census data indicated that there were 2762 housing units in Annotto Bay. The predominant material of construction that is the outer wall is concrete blocks which accounted for 48.1 per cent of all housing units (1329). The use of wood represented 38.8 per cent of the total (1074) while the combination of wood and concrete accounted for 10.8 per cent. Wattle/adobe and other type of outer wall materials accounted for less than 1 per cent.

Of the 3197 of total number of dwellings by tenure, 45% accounted for those that were owned, leased and rent free consisted of 13% and squatting a mere 1% and 43.9% being that for those dwellings that were rented as illustrated in Figure 5 below.

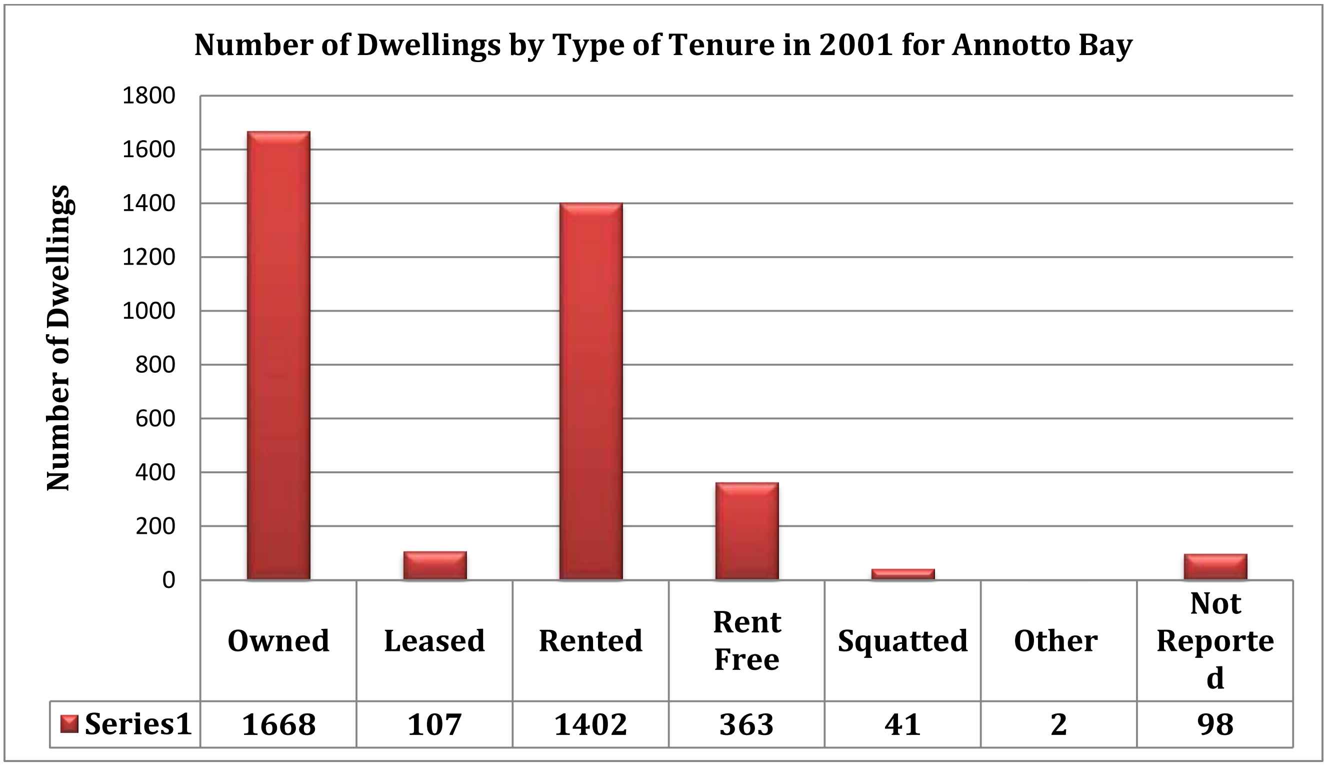


Figure 7: Number of Dwellings by Tenure

2.3.2 Economy and Employment

Annotto Bay is primarily an agricultural economy. However, the closing of sugar and downsizing of banana plantations, the community and by extension the parish of St. Mary has experienced the devastating decline in employment. This has resulted in serious economic decline.

Jamaica Producers is one of the main employers in Annotto Bay. Two subsidiary companies - St. Mary Banana Estate and JP Tropical Foods (Banana Chips Company) employs approximately 360 persons, 60 per cent of which are directly from the Annotto Bay community. The St. Mary Banana Estate produces bananas for both local market and export market.

3.0 Hazard Assessment

Hazard assessment is an essential first step of the risk assessment process. It involves gathering and analyzing data on meteorological, hydrological and geological hazards in terms of their nature, frequency and magnitude. Hazard assessment is characterized by triggering factors, degree of severity, spatial occurrence, duration of the event and their relationship. Three (3) main hazard types are considered within the scope of this project which includes:

1. Storm surge
2. Riverine Flood
3. Earthquakes

A detailed description of the methodologies, data used and hazard maps are presented in subsequent sections.

3.1 Storm Surge Assessment

Storm surge is an increase in water levels generated by a storm, which is above the normally expected astronomical tides. This rise in water level can cause extreme flooding in coastal areas particularly when storm surge coincides with normal high tide, resulting in storm tides which can reach levels of up to 20 feet or above mean sea level.

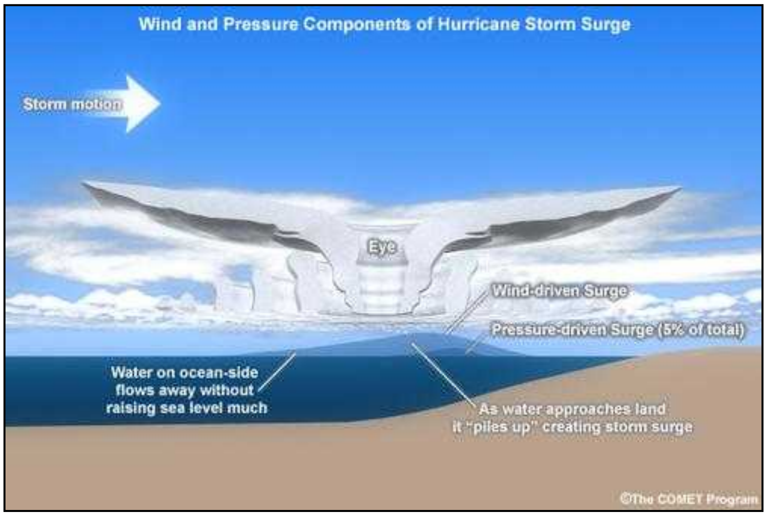


Figure 8: Wind and Pressure Components of Hurricane Storm Surge

3.1.1 Methodology for Storm Surge Mapping

Topographic survey – the survey was conducted to establish the existing shoreline and the back of beach elevations, a rapid topographic survey was conducted which extended from the shoreline through the residential developments to the north coast highway. Topographic data points were gathered relative to mean sea level (msl) by surveying the shoreline and making correction for tidal fluctuations using the British Admiralty Tidal Predictions for Port Antonio. The topographic surveys were supplemented with elevation data from the NLA 12:500 dataset from which a digital terrain model was created to represent the actual ground surface.

Bathymetric Data- bathymetric data forms the basis for wave transformation modeling and storm surge modeling to a lesser degree. Understanding the movement of currents along the seafloor aids in the prediction of wave intensity and direction on the shoreline. Mapping of the offshore shelf was carried out by the Marine Geology Unit, UWI; Mona with the assistance of local Fishermen to a maximum depth of 18 m. The data collection was collected using a depth sounder and a hand held GPS device. GPS positions and their corresponding depths were recorded which was digitally analysed using GIS to produce submarine contours of the bay. This contour data was supplemented by points and contours from the British Admiralty Chart 255-Eastern Approaches to Jamaica. The contours and points were used to create a digital terrain of the seafloor from shoreline to deep water. The bathymetry of the area north of the project site has a fairly constant drop off to the edge of the continental shelf which ends 0.75km offshore at approximately 20m depth.

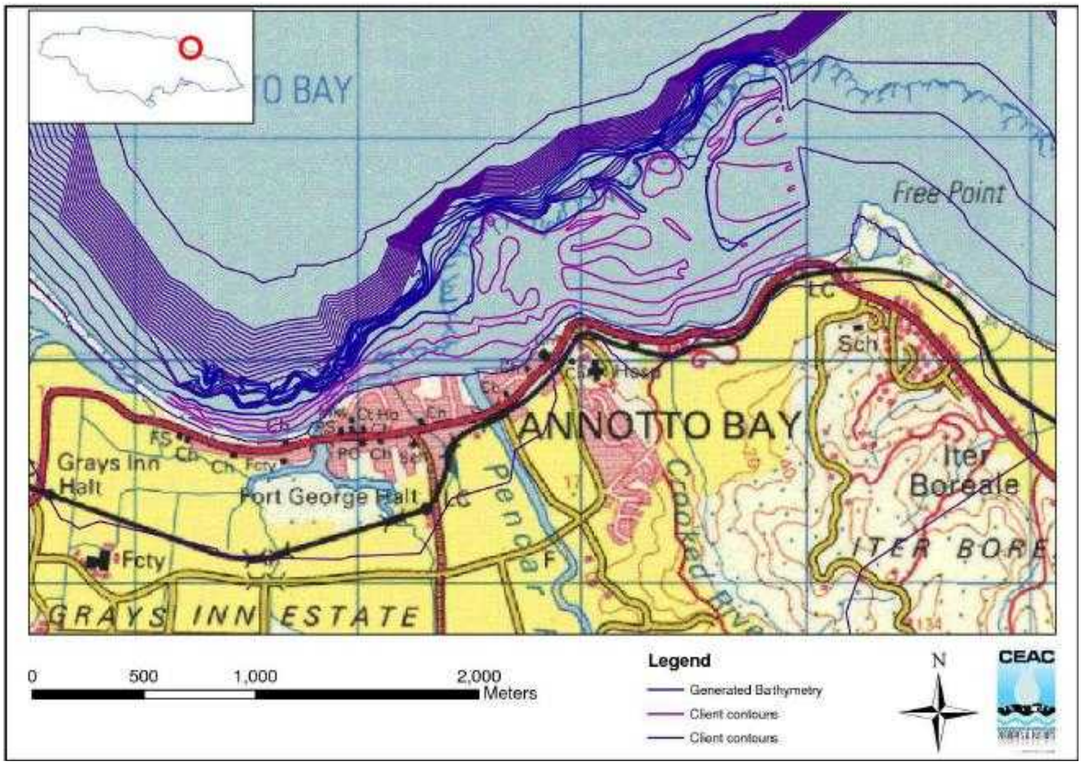


Figure 9: **Bathymetric features north of Project site**

Storm surge Modeling – The deepwater climate offshore Annotto Bay was first defined to determine wave parameters that would influence the coastline of the community as shown in Figure 9. The National Hurricane Center (NOAA) database of hurricane track data in the Caribbean Sea was then utilized to carry out a hindcast, followed by a statistical analysis to determine the hurricane waves, wind and set-up conditions. The database of hurricanes, dating back to 1886, was searched for storms that passed within a 300km radius from the site. The following procedure was carried out:

1. Extraction of Storms and Storm Parameters from the historical database. A historical database of storms was searched for all storms passing within a search radius of 300km radius of the site.
2. Application of the JONSWAP Wind-Wave Model. A wave model was used to determine the wave conditions generated at the site due to the rotating hurricane wind field. This is a widely applied model and has been used for numerous engineering problems. The model computes the wave height from a parametric formulation of the hurricane wind field.

3. Application of Extremal Statistics. Here the predicted maximum wave height from each hurricane was arranged in descending order and each assigned an exceedence probability by Weibull's distribution.

4. A bathymetric profile from deepwater to the site was then defined and each hurricane wave transformed along the profile. The wave height at the nearshore end of the profile was then extracted from the model and stored in a database. All the returned nearshore values were then subjected to an Extremal Statistical Analysis and assigned exceedance probabilities with a Weibull distribution.

The results of the search indicated the site's overall vulnerability and in summary:

- 78 hurricane systems came within 300 kilometers of the project area
- 2 of which were classified as catastrophic (category 5)
- 18 were classified as extreme (Category 4)

The most frequent hurricane waves have been noted to approach from an easterly direction followed closely by the North-easterly winds as shown in Table 3.

The maximum and minimum confidence levels showed increased variance from the return values as the return period increases. The confidence limits for the wave setups showed an average variance of less than 0.36m between return value and the maximum and minimum levels for the 100 year period. This is reasonable given that the source of data covers 125 years. **Wave setup**[3] occurs when waves continually break onshore and the water from the runup piles up along the coast because it can't get out back to sea. The water level therefore rises as hurricane approaches, especially since the waves become larger and more water is pushed onshore.

[3] NOAA National Hurricane Centre.(n.d.). Introduction to Storm Surge. Retrieved from: http://www.nws.noaa.gov/om/hurricane/resources/surge_intro.pdf

Table 3: Extremal Storm Surge (m) Predictions for the Site Along the Profile from Shoreline to Deepwater for All Directional Waves Possible for Annotto Bay

Return Period	Total setup (m)								
	All	SW	W	NW	N	NE	E	SE	S
1		0.00	0.00	NaN	0.00	NaN	NaN	0.00	0.00
2	NaN	0.00	0.00	NaN	0.00	NaN	0.55	0.00	0.00
5	0.71	0.00	0.00	0.60	0.00	0.69	0.95	0.00	0.00
10	0.88	0.00	0.00	0.70	0.00	0.80	1.19	0.00	0.00
20	1.05	0.00	0.00	0.80	0.00	0.91	1.41	0.00	0.00
25	1.10	0.00	0.00	0.83	0.00	0.94	1.48	0.00	0.00
50	1.27	0.00	0.00	0.92	0.00	1.05	1.68	0.00	0.00
75	1.37	0.00	0.00	0.97	0.00	1.11	1.80	0.00	0.00
100	1.44	0.00	0.00	1.01	0.00	1.15	1.88	0.00	0.00
150	1.54	0.00	0.00	1.06	0.00	1.21	2.00	0.00	0.00
200	1.61	0.00	0.00	1.09	0.00	1.26	2.08	0.00	0.00

In addition, static storm surge was also investigated in the analysis for all major components of storm surge. The phenomena considered were:

- Wave breaking and shoaling
- Wind set-up
- Refraction
- Tides
- Global Sea Level Rise (over a 50 year project life)
- Inverse Barometric Pressure Rise

The storm surge with run-up was chosen as the benchmark model for determining the 10, 25, 50 and 100 year period storm surge levels for Annotto Bay. Return period indicates the period in years in which the hazards is likely to occur based on historic events. The surge inundation zones were mapped and plotted over satellite imagery for the return periods aforementioned in a GIS environment.

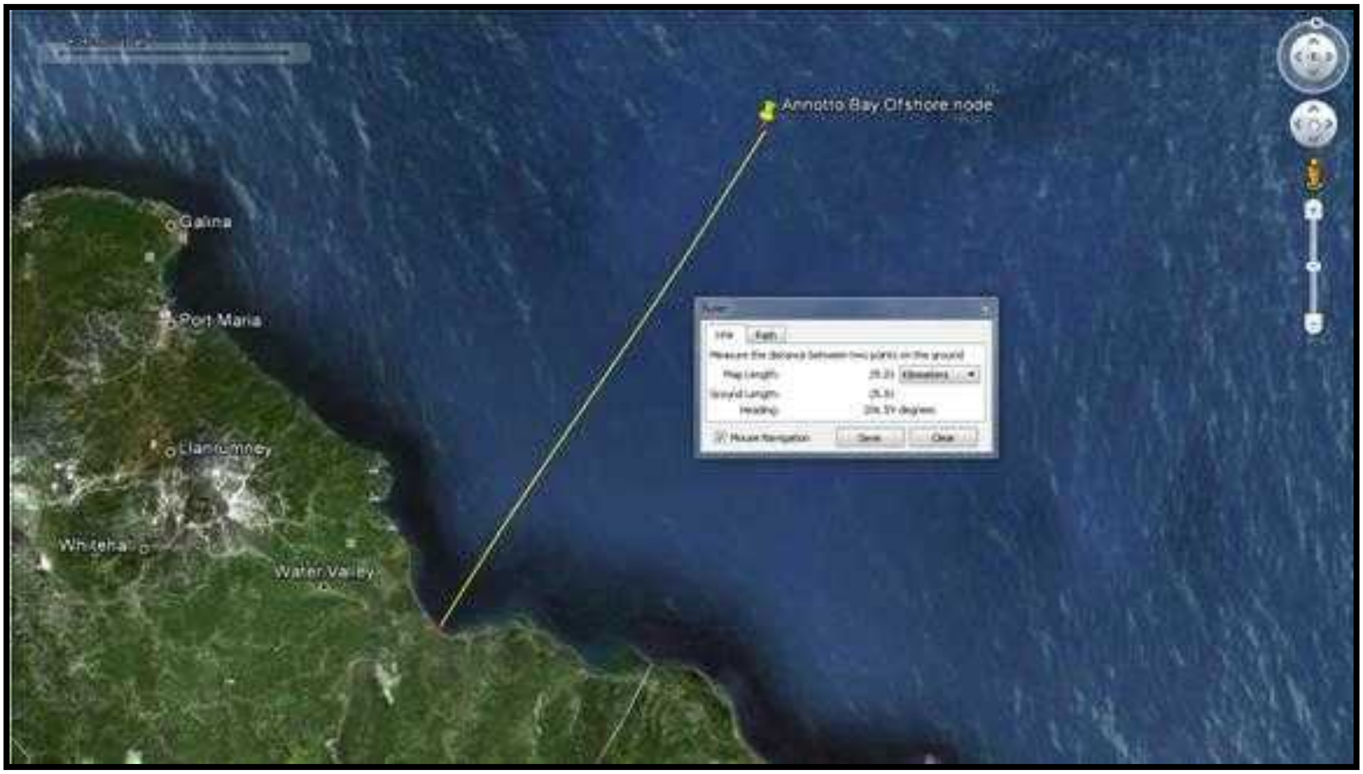


Figure 10: Location of Offshore Point Used for Extremal Analysis

Anecdotal Evidence of Storm Surge - Anecdotal evidence of past storms was collected to aid in the verification of a storm surge model for the area. Such evidence was also used to generate an estimate of the return period for actual storm surge versus estimated. Interviews were conducted with available residents of Annotto Bay with living first hand memory of hurricane events. Overall, twenty (20) interviews were done with residents with an average age of 52.2 years and living an average of 41 years in Annotto Bay. The respondents recalled 5 storms, including: Allen (1980), Gilbert (1988), Ivan (2004), Dennis (2005) and Gustav (2008). The resulting average setup for each storm is summarized (See Annex B).

Two storms were eliminated from the average observed setups and comparisons to model results, as they conflicted with what the general understanding of what should have occurred. Ivan passed on the southern coast of Jamaica and could not have generated significant storm surge in Annotto Bay as some respondents had reported. Hurricane Gilbert passed on over the island and was also eliminated for the same reason.

The observed setups were subjected to extremal statistical analysis to estimate the return period of the setups experienced. The statistical tool used was the Weibull function which is widely used for this type of extremal data analysis due to it having three variables which enables it to obtain a better fitted curve those others which have only two variables.
One factor that was unaccounted for in the model prediction however is the effect of wave run-up which will inevitably increase the water levels. This parameter would not have been easily differentiable to the observers and would have thus been a part of what was observed. It is against this background that wave run-up was determined and added to the storm surge elevations.

The Software Programme Cresswin was utilized to estimate the runup. This software uses the model for wave run-up on smooth and rock slopes of coastal structures according to Van der Meer et al[4]. The Estimated run-up ranges from 0.705m to 1.441m for the 5 to 100 year hurricane waves and were added to the model predicted storm surge results.

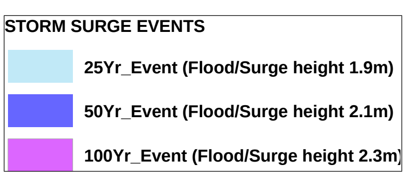
3.1.2 Limitations

The main limitation is the availability of data, specifically detailed topographic data which would refine storm surge elevations, thereby allowing for the best possible simulation of inundation. The accuracy, therefore of the modeled storm surge inundation is limited by the data that was available.

3.1.3 How to Read Map

The storm surge hazard map shows the following:

- Extent of inundation for 25, 50, 100 year return period. Each colour of inundation area represents different return period.



[4] Van Deer Meer, J. & C.J.Stam.(1999). Wave Run-Up on Smooth and Rock Slopes of Coastal Structures. ASCE Journal of WPC & OE. Vol. 118, pp 534-550.

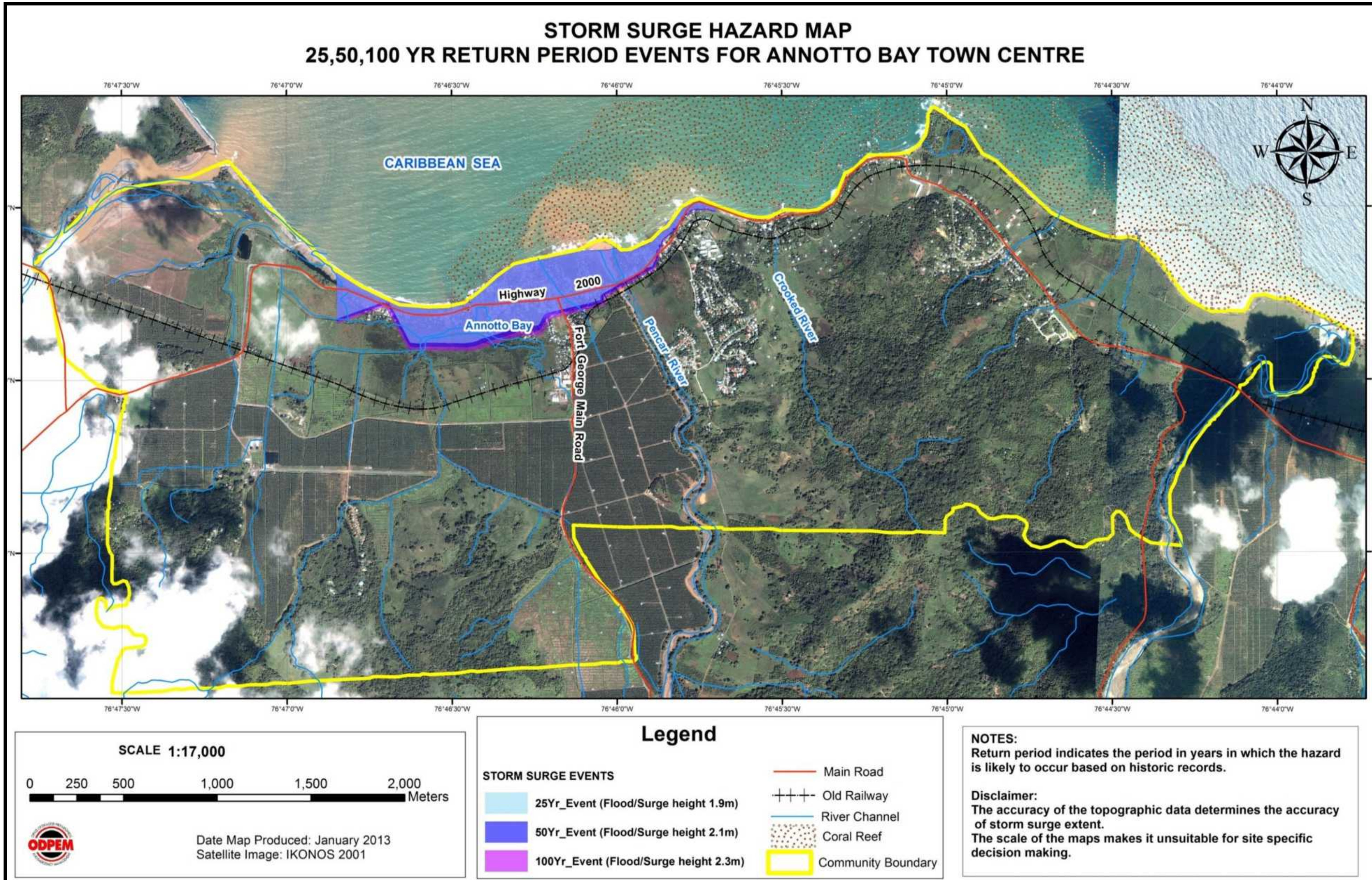


Figure 11: **Storm Surge Hazard Map for Annotto Bay Town Centre (Master Sheet)**

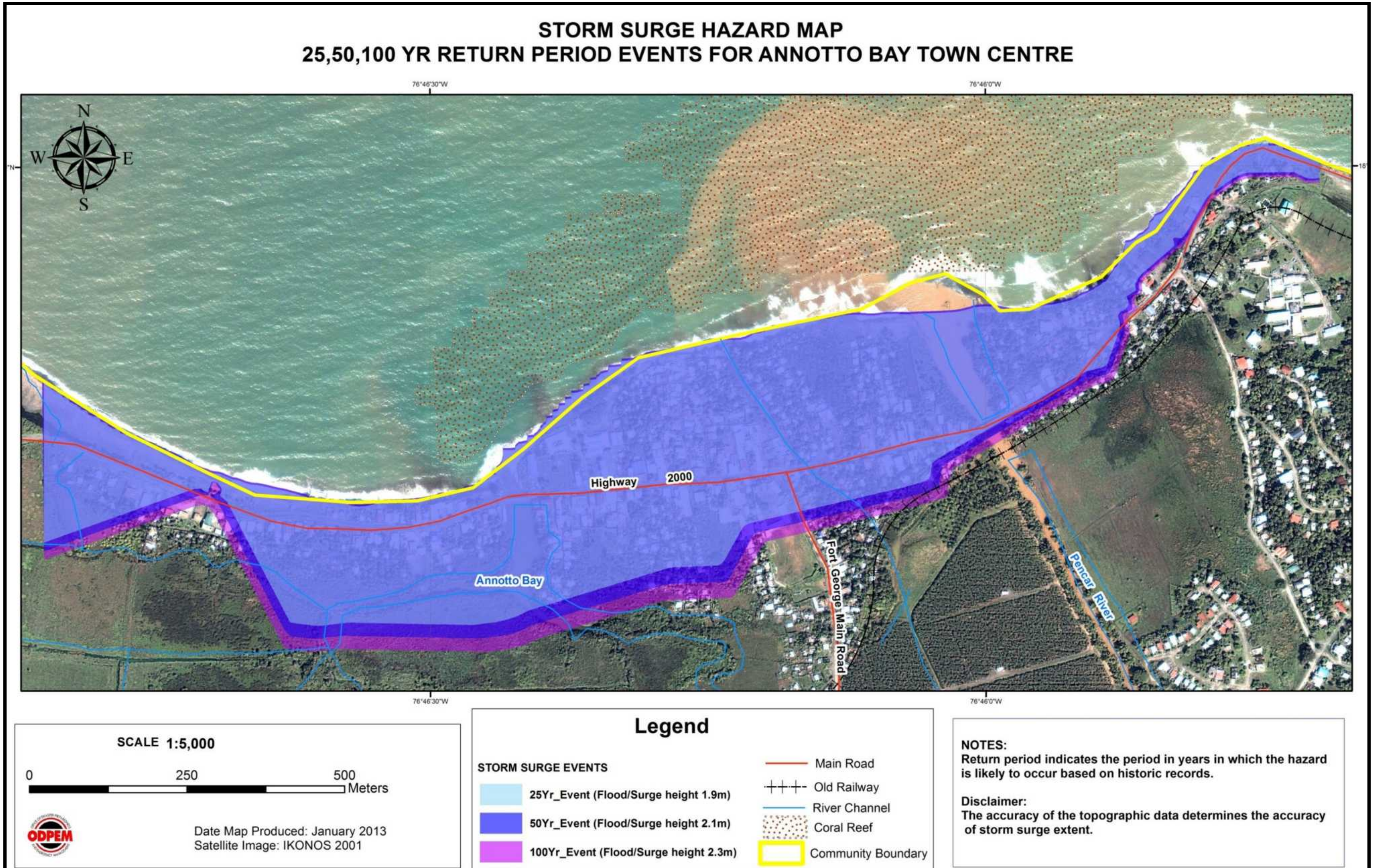


Figure 12: **Storm Surge Hazard Map for Annotto Bay**

3.1.4 *Analysis of Storm Surge Hazard Assessment*

The inundation boundaries for the 25, 50 and 100 year flood level contours were plotted over the digital terrain model for Annotto Bay. Storm surge inundation maps for different return periods were overlaid on IKONOS imagery for the area.

Annotto Bay is a relatively flat coastal area and as such storm surge can penetrate well inland from the coastline as is shown in Figures 10 and 11. The predicted storm surge height or flood depth with run-up is estimated at 1.9m, 2.1m, 2.3m for the 25, 50, 100 year return period, respectively. Wave run-up occurs when a wave breaks and the water is propelled onto the beach. The maximum extent of flooding inland for each corresponding surge height was estimated from the shoreline at 312m, 378m and 407m inland for the 25, 50 and 100 year return period. It can be seen (Figure 11) that most of the developments from the coastline to just south of the highway, particularly the town proper are susceptible to flooding. The approximate inundation area that is, the total area expected to be affected by storm surge ranges between 39333 – 474816.4 square metres. West of the town, moving towards Iterboreale susceptibility to inundation is lower owing to the slightly higher elevation in this zone. Table 4 depicts the approximate inundation area for each return period with corresponding flood depth/surge height.

Table 4: Estimated Inundation Area for Each Storm Surge Return Period

Return Period (yr)	Storm Surge Height (m)	Max. Storm Surge Distance Inland (m)	Approximate Inundation Area (sq. metres)
25	1.9	312	3.93
50	2.1	378	436.88
100	2.3	407	474.82
TOTAL			

It must be noted that the storm surge model for Annotto Bay compares well with the model results for Hurricane Allen in 1980, a category 4 hurricane which passed within 30 miles of

Port Antonio. That is, the modeled maximum storm surge distance inland would inundate the same areas as Hurricane Allen; however the expected extent of flooding from the model covers a slightly larger area.

There are two major river systems in Annotto Bay which are subjected to storm surge penetration- Pencar and Annotto Rivers as well as Motherford Drain. Research has shown that because of less friction over land, the storm surge would penetrate 10-15% more distance inland through the river systems, which is a general accepted assumption globally. This means that areas along the banks of the rivers – Cane Lane and Dump could experience greater flood depth.

3.2 Coastal Erosion

3.2.1 Methodology for Shoreline Erosion

As aforementioned, the hurricane waves originating from the East are the most severe of all the directions investigated. The eastern waves are however not expected to significantly impact the site as much the North easterly profile would, due to the Northern projection of the land by Iter Boreale which is located east of Annotto Bay.

Sediments and Grain Size Analysis - It was necessary to determine the representative grain size on the shoreline in order to understand how the beach will react to the hurricane waves. Four (4) surface samples (Figure 12) were collected from the project beach face and analyzed to determine the representative grain size and distribution. The grain size analysis was done using the unified classification which is widely used for classification of granular material. The samples had median grain sizes varying from 0.413mm to 1.465mm in diameter. The classification of these samples therefore varied from medium to very coarse sand.

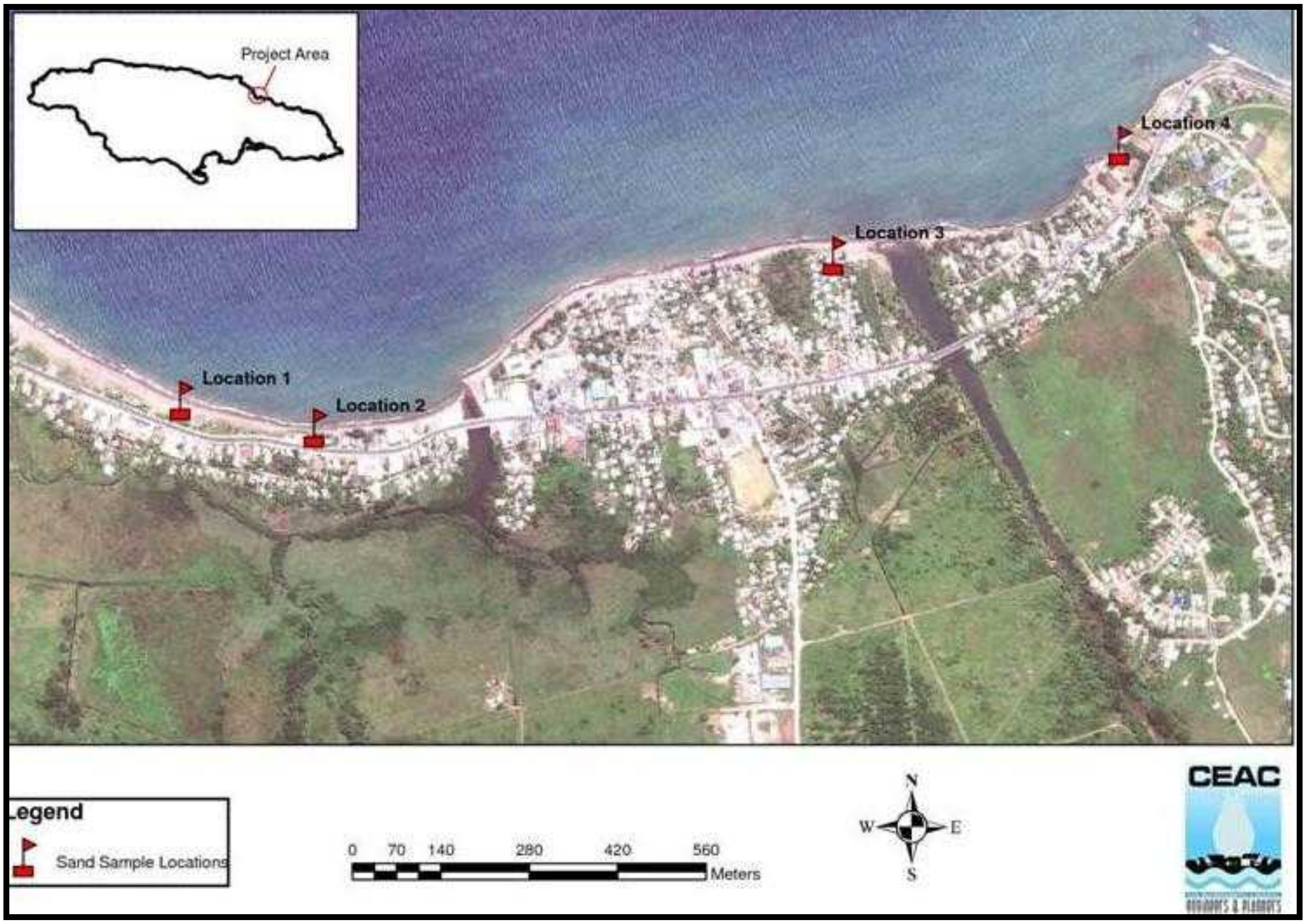


Figure 13: Sediment Sample Locations

Model Description and Input - SBEACH is an empirically based numerical model for estimating beach and dune erosion due to storm waves and water levels. The magnitude of cross-shore sand transport is related to wave energy dissipation per unit water volume in the main portion of the surf zone. The direction of transport is dependent on deep water wave steepness and sediment fall speed. SBEACH is a short-term storm processes model and is intended for the estimation of beach profile response to storm events. Typical simulation durations are limited to hours to days (1 week maximum).

Profiles were cut from deepwater to land up to a maximum elevation of approximately six (6) metres at four locations (Figure 13) from northern and north easterly directions spanning the entire project shoreline. The wave data from the deep water hurricane model was utilized for this analysis to represent the most vulnerable directions. Table 5 shows the

10-100 year return period wave characteristics utilized in the model and the input parameters for the model for each profile. Other input parameters included the sediment grain size on the beach face and storm duration.

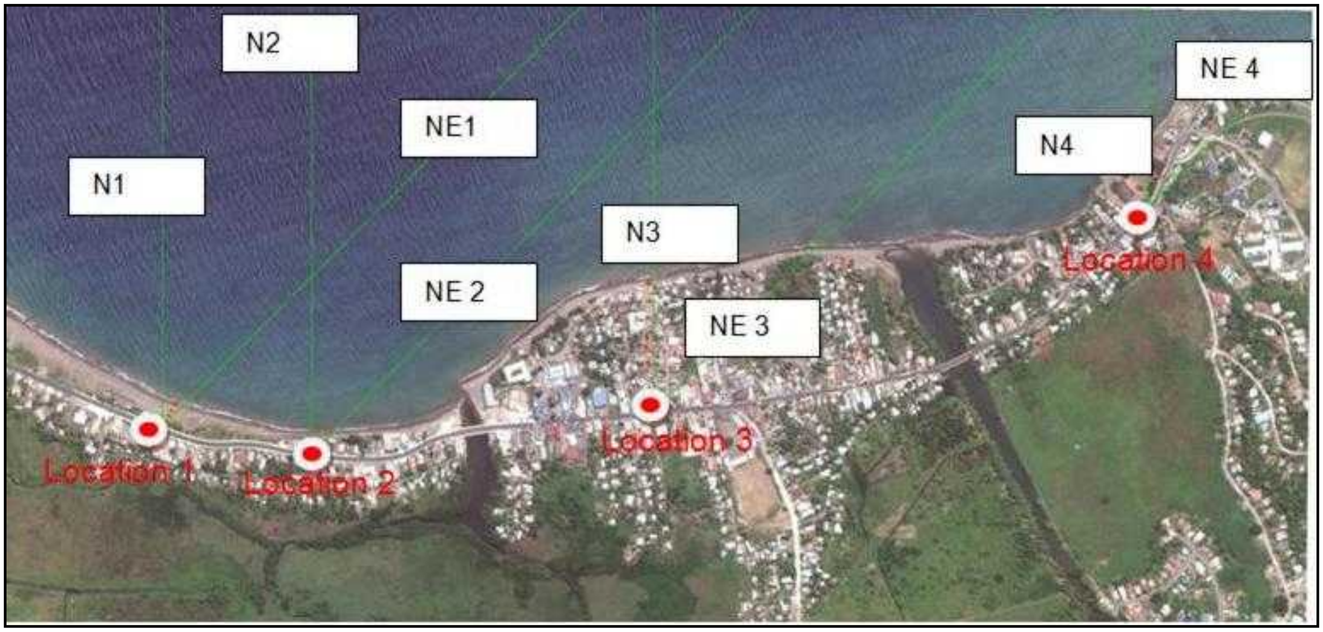


Figure 14: Locations along Shoreline where Profiles were cut

Table 5: SBEACH Input Parameters for 10, 50, 100 Year Return Storm

Locations	10	50	100
Directions	**All**	**All**	**All**
Input Parameters			
Hs (m)	**5.80**	**6.68**	**6.90**
Tp (s)	**12.1**	**12.84**	**13.10**
Deepwater storm surge	**0.88**	**1.27**	**1.44**

Hs – Wave height   Tp -

3.2.2 How to Read Map

The Erosion map has been prepared based on a 10 year storm event. The red polygon predicts the shoreline erosion rate for Annotto Bay during a 10 year storm.

3.2.3 Analysis for Shoreline Erosion

The maximum wave heights estimated at the shoreline as a result of wave transformation varies from 1.8m to 3.4m from the 10 to 100 year storm (See Table 6). These wave heights arriving at the shoreline possess the potential for serious damage to the beach and to structures behind the beach.

The shoreline is predicted to erode between 24 to 38 metres during a 10 year storm from the north eastern and northerly directions in locations one (1) and three (3), respectively. No erosion was predicted for the higher return periods because the wave heights arriving at the shoreline are not significantly different, but the differences in the setups are much larger. This has resulted in the waves exerting more force on the shoreline and causing erosion.

Figure 15: Shoreline Erosion Hazard Footprint

3.3 Riverine Flood Assessment

As indicted earlier in Chapter 2, the community of Annotto Bay lies between the mouth of the Pencar and Annotto Rivers. Hence, the community is also very flat (See Figure 15) and is therefore repeatedly affected by flooding once capacity of river networks is breached.

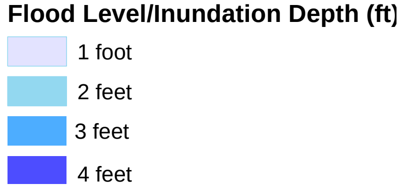
The flood assessment is based on the last major flood event that happened in the community. In 2001, unstable weather conditions associated with Tropical Storm Michelle produced over 1000 millimetres of rainfall on both 28 and 29 October, 2001 for the parish of St. Mary. This resulted in the main rivers, namely Annotto and Pencar Rivers breaching their banks which resulted in widespread flooding of the community. Figure shows the spatial extent of flooding and flood levels in the community.

The Water Resources Authority (WRA) led the data collection process primarily mapping of inundated areas as well as from direct observations and interviewing residents. Information was also obtained from recording station which included estimation of peak flows based on recorded hydrographs and collection of rainfall data from data loggers. Return periods were calculated using the "Regional Flood Frequency Analysis in Jamaica".

3.3.1 How to Read Map

The storm surge hazard map shows the following:

- Extent of inundation associated with the 2001 flood event. Each colour of inundation area different level of flood depth.



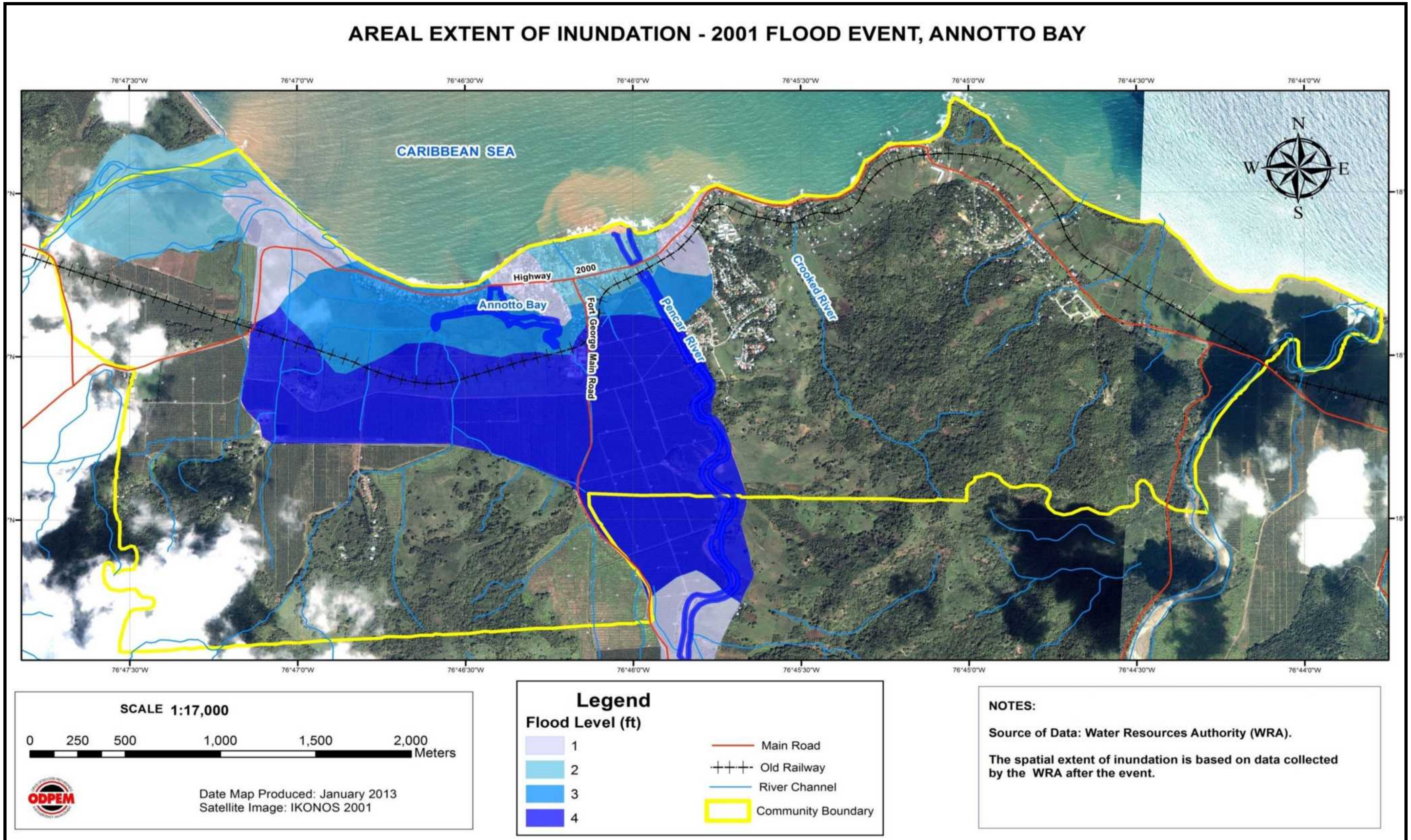


Figure 16: Flood Hazard Map showing inundation during the 2001 Flood Event in Annotto Bay

3.3.2 *Analysis of Flood Hazard Assessment*

The areal extent of inundation as illustrated in Figure 15 covered approximately 436.0 hectares of land across Annotto Bay from the Pencar River Main Bridge in the west of the Fire station in the east of the community. The entire town centre was inundated to a depth ranging from 1 to 4 feet. The areas which experienced greater flood depth are located on the flood fringe which includes the agricultural lands belonging to St. Mary Banana Estate or JP Food and the buildings along the Annotto and Pencar Rivers. Assessment by the WRA estimated the flow velocity was to be less that 0.3 m/s which resulted in limited or no structural damage.

3.4 Seismic Assessment

3.4.1 *Geology of Annotto Bay*

The surficial geological units in the study area are alluvial deposits overlying White Limestone of the Gibraltar- Bonny Gate Formation and the Richmond Formation. The alluvial deposits occur along the coast and lower reaches of the major rivers that empty along the coastal flats in the town of Annotto Bay. The deposits range from carbonaceous to silica rich sands with abundant shell fragments that vary in thickness and reaches up to 70 metres in the western section of the town in the Aqualtavale area.

The Richmond Formation outcrops towards the western section of the town. It is composed of a series of well bedded grey to brown–weathering alternating calcareous sandstones, siltstones and mudstones with occasional thin beds of limestone and massive conglomerates. The Gibraltar- Bonny Gate Formation outcrops towards the eastern end of the town and can be described as a series of evenly bedded chalky limestone with occasional bioclastic layers.

Faulting is the dominant structural feature in the area with the longest fault lines being two unnamed major faults showing a north-south trend which appears at the eastern limit of the Wagwater Formation (Figure 8). The Vere-Annotto Bay fault is also a major fault in the study area with a SW-NE trend; however, there are no surficial expressions of this fault line. Furthermore, several faults with a general east-west trend are also dominant in the area. Minor surface expression of the fault is shown in the area and the boundary of the more easterly Gibraltar Formation and Richmond Formation is marked by an east west trend minor fault that borders the slopes surrounding the town from the northerly coastal flats.

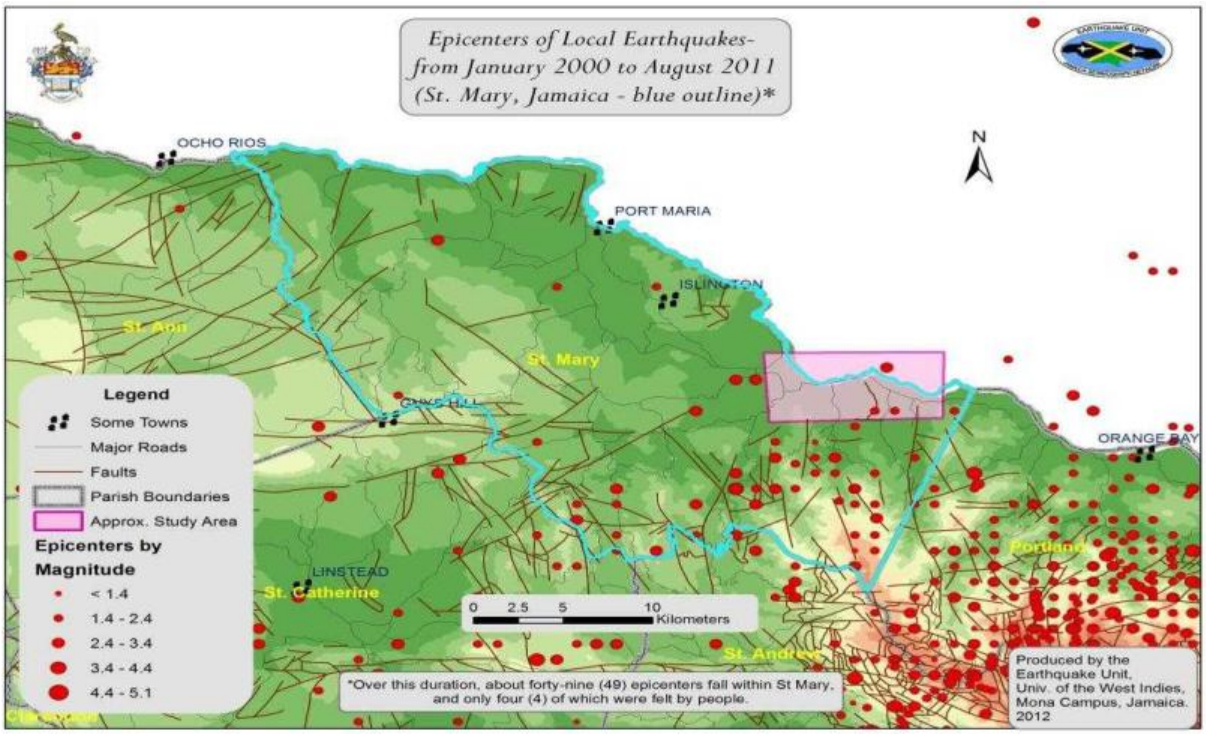


Figure 17: Major Faults and Epicentres of Earthquake Events (2000-2011) in relation to Study Area.

3.4.2 Methodology for Seismic Assessment

3.4.2.1 Nakamura or H/V Method

Two approaches were undertaken to assess the seismic vulnerability of Annotto Bay. The first technique known as the Nakamura or H/V method is a technique originally proposed by Nogoshi and Igarashi (1971) and made wide-spread by Nakamura et al. (1983), and entails estimating the ratio between the Fourier amplitude spectra of the horizontal (H) to vertical (V) components of ambient noise vibrations recorded at one single station. For H/V measurements these 3 components of ground motion are required. The three- channel portable seismograph measures 3 signals: North-South, East-West and Vertical at each site.

The result of this survey identifies the fundamental frequency/period of the site. Ambient vibration recordings combined with the H/V spectral ratio technique have been proposed to help in characterising local site effects. As it is well known, occurrence of earthquake damage depends upon strength, period (time) and duration of seismic motions and these parameters are strongly influenced by seismic response characteristics of surface ground and structures.

The H/V technique has been frequently adopted in seismic microzonation investigations. This technique is most effective in estimating the natural frequency of soft soil sites when there is a large impedance contrast with the underlying bedrock. The method is especially recommended in areas of low and moderate seismicity due to the lack of significant earthquake recordings, as compared to high seismicity areas. Site effects associated with local geological conditions constitute an important part of any seismic hazard assessment. Many examples of catastrophic consequences of earthquakes have demonstrated the importance of reliable analyses procedures and techniques in earthquake hazard assessment and in earthquake risk mitigation strategies.

The software known as Geopsy has been used to process raw field data into a H/V spectral ratio from any type of vibration signals (ambient vibrations, earthquake). A typical output from this assessment is shown in the Figure 16 below.

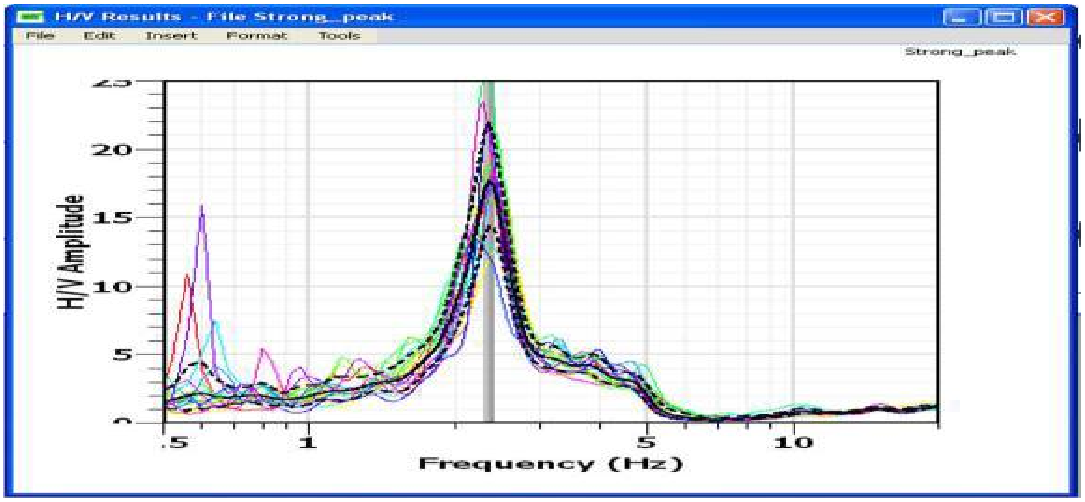


Figure 18: Typical output of the processed 3-component ambient ground motion signal to determine the fundamental frequency of a site.

In Figure 18, the black curve represents H/V geometrically averaged over all coloured individual H/V curves. The two dashed lines represent H/V standard deviation. The grey area represents the averaged peak frequency and its standard deviation. The frequency value is at the limit between the dark grey and light-grey areas. The period is calculated by determining the inverse value of the fundamental frequency.

3.4.2.2 Rapid Visual Screening Method

The second technique is an assessment of the community by Rapid Visual Screening (RVS, 2002), a methodology developed by the United States Federal Emergency Management Agency (FEMA). The RVS has been used by FEMA as a guideline to assess the structural integrity of buildings and this methodology has been adopted for use in India, (Sadat et. al, 2010) Turkey (Yakut 2004), Oregon (Wang and Goettel 2007) to assess the seismic vulnerability of town and cities. The RVS has been developed for use by a range of construction professionals including building officials and inspectors, and government agencies and private-sector building owners to identify, inventory, and rank buildings that are potentially seismically hazardous. The RVS uses a methodology based on a "sidewalk survey" and uses a Data Collection Form specific to the level of seismicity of the country or region, i.e. Low, Moderate or High. The person conducting the survey completes this form assigning scores based on the parameters examined or applicable to the type of building as shown in Table 6. This assessment is based on visual observation of the building from the exterior, and if possible, the interior. The Data Collection Form includes space for documenting building parameters, identification information, including its use and size, a photograph of the building, sketches, and documentation of pertinent data related to seismic performance, including the summation of a numeric seismic hazard score for the building based on the parameters used by the FEMA guideline to arrive at a final score of the building, as illustrated in Figure 10.

Although RVS is applicable to all buildings, its principal purpose is to identify (1) older buildings designed and constructed before the adoption of adequate seismic design and detailing requirements, (2) buildings on soft or poor soils, or (3) buildings having performance characteristics that negatively influence their seismic response. The intended use of the RVS procedure is to screen a population of buildings on the basis of a cut-off value after a final score is determined. The final score results separates the buildings into two categories:

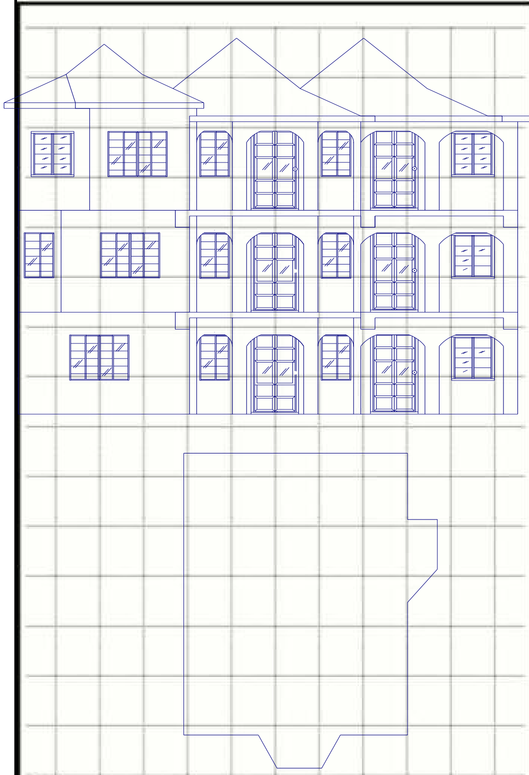
- those that are expected to have acceptable seismic performance
- those that may be seismically hazardous and should be studied further

Once identified as potentially hazardous, such buildings should be further evaluated by a professional engineer experienced in seismic design to determine if, in fact, they are seismically hazardous.

Rapid Visual Screening of Buildings for Potential Seismic Hazards
FEMA-154 Data Collection Form

MODERATE Seismicity

Address: ______________________
______________________ Zip ______
Other Identifiers ______________________
No. Stories ______________ Year Built ______
Screener ______________ Date ______
Total Floor Area (sq. ft.) ______________
Building Name ______________________
Use ______________________



Scale:

OCCUPANCY			SOIL	TYPE						FALLING HAZARDS			
Assembly Commercial Emer. Services	Govt Historic Industrial	Office Residential School	Number of Persons 0 – 10 11 – 100 101-1000 1000+	A Hard Rock	B Avg. Rock	C Dense Soil	D Stiff Soil	E Soft Soil	F Poor Soil	☐ Unreinforced Chimneys	☐ Parapets	☐ Cladding	☐ Other:

BASIC SCORE, MODIFIERS, AND FINAL SCORE, S

BUILDING TYPE	W1	W2	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RC SW)	S5 (URM INF)	C1 (MRF)	C2 (SW)	C3 (URM INF)	PC1 (TU)	PC2	RM1 (FD)	RM2 (RD)	URM
Basic Score	5.2	4.8	3.6	3.6	3.8	3.6	3.6	3.0	3.6	3.2	3.2	3.2	3.6	3.4	3.4
Mid Rise (4 to 7 stories)	N/A	N/A	+0.4	+0.4	N/A	+0.4	+0.4	+0.2	+0.4	+0.2	N/A	+0.4	+0.4	+0.4	-0.4
High Rise (>7 stories)	N/A	N/A	+1.4	+1.4	N/A	+1.4	+0.8	+0.5	+0.8	+0.4	N/A	+0.6	N/A	+0.6	N/A
Vertical Irregularity	-3.5	-3.0	-2.0	-2.0	N/A	-2.0	-2.0	-2.0	-2.0	-2.0	N/A	-1.5	-2.0	-1.5	-1.5
Plan Irregularity	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
Pre-Code	0.0	-0.2	-0.4	-0.4	-0.4	-0.4	-0.2	-1.0	-0.4	-1.0	-0.2	-0.4	-0.4	-0.4	-0.4
Post-Benchmark	+1.6	+1.6	+1.4	+1.4	N/A	+1.2	N/A	+1.2	+1.6	N/A	+1.8	N/A	2.0	+1.8	N/A
Soil Type C	-0.2	-0.8	-0.6	-0.8	-0.6	-0.8	-0.8	-0.6	-0.8	-0.6	-0.6	-0.6	-0.8	-0.6	-0.4
Soil Type D	-0.6	-1.2	-1.0	-1.2	-1.0	-1.2	-1.2	-1.0	-1.2	-1.0	-1.0	-1.2	-1.2	-1.2	-0.8
Soil Type E	-1.2	-1.8	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6

FINAL SCORE S

COMMENTS

Detailed Evaluation Required
YES NO

* = Estimated, subjective, or unreliable data
DNK = Do Not Know

BR = Braced frame
FD = Flexible diaphragm
LM = Light metal

MRF = Moment-resisting frame
RC = Reinforced concrete
RD = Rigid diaphragm

SW = Shear wall
TU = Tilt up
URM INF = Unreinforced masonry infill

Figure 19: Completed assessment of a building using the RVS Methodology

3.4.2.3 Parameters Considered in RVS

The parameters used in screening buildings to determine the total numerical score of a building includes the seismic hazard intensity, building type, height of the building, soil type in the foundation, plan and vertical irregularity of the building, conformity to the seismic building code in the design (see Table 7 for discussion of the properties of these modifiers).

Each Hazard Intensity Form (Low, Moderate, or High) has separate scoring values for each building type and each score vary for each modifier (parameter) for each building type. The building type is assigned an initial basic score which is in fact related to its lateral load resisting structural system and earthquake performance and then additional modifying scores (only those specific to the building and soil type) are added or subtracted from the basic score to arrive at a final score for each building in the assessment.

Table 8 shows the different soil types (with explanation of the geophysical characteristics) that defines the soil modifier in the different sections of the community.

Table 6: FEMA Classification Building Type considered by the RVS Procedure

Building Code	Building Description	Building Code	Building Description
W1	Light wood-frame residential and commercial buildings smaller than or equal to 5,000 square feet	C2	Concrete shear-wall buildings
W2	Light wood-frame buildings larger than 5,000 square feet	C3	Concrete frame buildings with unreinforced masonry infill walls
S1	Steel moment-resisting frame buildings	PC1	Tilt-up buildings

Building Code	Building Description	Building Code	Building Description
S2	Braced steel frame buildings	PC2	Precast concrete frame buildings
S3	Light metal buildings	RM1	Reinforced masonry buildings with flexible floor and roof diaphragms
S4	Steel frame buildings with cast-in-place concrete shear walls	RM2	Reinforced masonry buildings with rigid floor and roof diaphragms
S5	Steel frame buildings with unreinforced masonry infill walls	URM	Unreinforced masonry bearing-wall buildings (Also made to include ***Wattle and Daub*** structures – building technique which utilizes a woven lattice of wood strips daubed with wet soil such as clay and straw.)
C1	Concrete moment-resisting frame buildings		

After a complete assessment is done of a building, a final score is obtained which determines the expected seismic performance of that building. The cut-off score and final score of the structure indicates if the building is seismically safe or unsafe. If unsafe detailed engineering assessment is required. The cut-off score used in this study for non-critical facilities was 2.5 which is a little higher than FEMA's typical score of 2.0. A greater score was chosen due to the fact that the study area is located in the section of the island that has highest frequency of seismic activities. Mathematically, a final score of 2.0 means an estimated 1% chance of collapse at the defined level of ground shaking in the area of the country where the building is located.

The scores are logarithmically related to the likelihood of complete structural damage (and collapse), but suffice it to say that a number above 2.5 means the building probably represents a low collapse risk in an extreme earthquake, and a number below 2.5 means the building is of enough concern to warrant a detailed seismic evaluation by a qualified structural engineer. One of the more difficult steps in the RVS procedure is determining the cut-off score, since it poses the question involving the cost of safety versus the benefits. In general, buildings which fall in the category of emergency services are normally given a cut-off score of 3 which indicates that the buildings with a score of 3 or more would have a 1 in 10^3 chance of receiving severe damage in the event of major earthquake. There are several factors to consider when selecting a cut-off score for a region. The present state of the country's economy is one factor that is considered when selecting a cut-off score. The economic stability of the country becomes relevant in the decision process because the higher the cut-off score the more likely for building's final score to fall below the threshold value. Structures which do not meet the cut-off score would therefore require a detailed evaluation to be done, which can be very costly as professional personnel with specialized equipment would be employed to determine the potential of seismic hazards (FEMA -154).

Table 7: Description of applicable modifiers used in scoring the performance of each building.

Modifier	Modifiers Description
Mid-Rise	4-7 Storeys
High-Rise	8 or more Storeys
Vertical irregularity	Hillside buildings, soft storeys, irregular shape in elevation
Plan irregularity	Buildings with re-entrant corners, buildings with good lateral-load resistance in one direction but not in the other; and buildings with major stiffness eccentricities in the lateral force- resisting system, L shaped, T-Shaped, U-shaped, large openings, Weak Link Between Larger Building Plan Areas
Pre-Code	buildings in high and moderate seismicity regions and is applicable if the building being screened was designed and constructed prior to the initial adoption and enforcement of seismic codes applicable

	for that building type
Post-Benchmark	Building designed and constructed after significantly improved seismic codes applicable for that building type (e.g., concrete moment frame, C1)
Soil Type	Score Modifiers are provided for Soil Type C, Type D, and Type E. The appropriate modifier should be circled if one of these soil types exists at the site

Table 8: Soil Type Definitions and Related Parameters

Soil Type Definitions	Related Parameters
Type A **(hard rock)**	Measured shear wave velocity (*vs)* **>** 5000 ft/sec.
Type B **(rock)**	*vs* between 2500 and 5000 ft/sec.
Type C **(soft rock and very dense soil)**	*vs* between 1200 and 2500 ft/sec, or standard blow count(*N)* **>** 50, or undrained shear strength (*su)* **>** 2000 psf.
Type D **(stiff soil)**	*vs* between 600 and 1200 ft/sec, or standard blow count (*N)* between 15 and 50, or undrained shear strength (*su)* between 1000 and 2000 psf.
Type E **(soft soil)**	More than 100 feet of soft soil with plasticity index (PI) > 20, water content (w) > 40%, and *su* **<** 500 psf; or a soil with *vs* ≤ 600 ft/sec.
Type F **(poor soil)**	Soils requiring site-specific evaluations:

3.4.3 Analysis of Seismic Assessment

3.4.3.1 H/V Field Data

There are 14 points where measurements were done in Annotto Bay (Figure 18). The readings were taken at approximately 500 metres between each point. Portable seismograph (Guralp 40 T) instrument was used to collect ambient ground motion. For each of these sites care was taken to ensure that the location had minimal noise (anthropogenic or natural). The Instrument was left to stand at each point on average 30 minutes so that good quality data could be collected over this period. Having calculated the fundamental frequency, the period (inverse of frequency) of each site was also calculated as shown in Table 9.

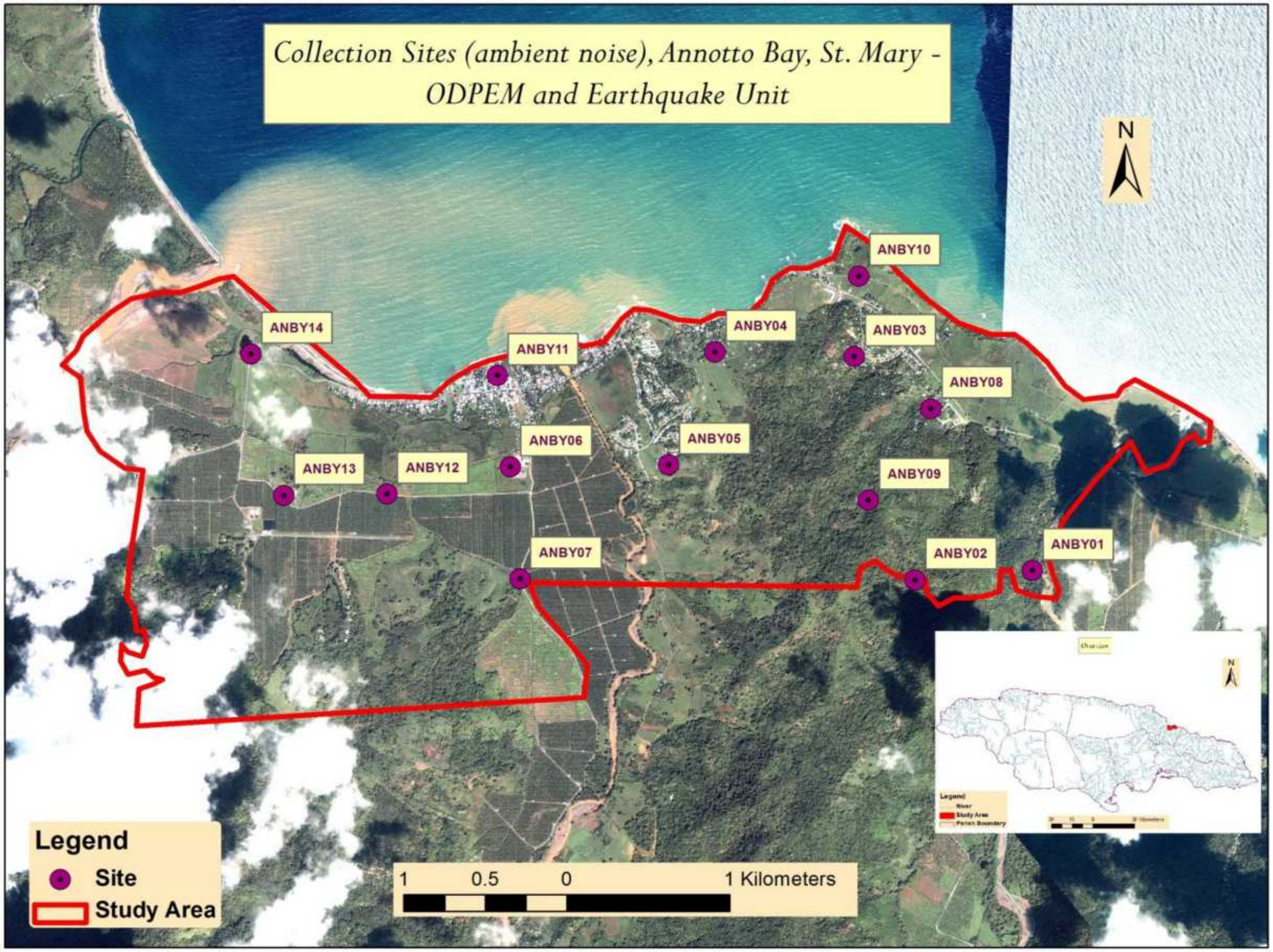


Figure 20: Location of areas where Site Effect Study was conducted in Annotto Bay

Table 9: Period of each site determined from the fundamental frequency of each H/V spectral ratio.

Site	Fundamental Frequency (Hz)	Period (s)
ANBY01	0.7	1.4
ANBY02	8	0.1
ANBY03	3	0.3
ANBY04	2	0.5
ANBY05	3.5	0.3
ANBY06	0.7	1.4
ANBY07	1.5	0.7
ANBY08	2	0.5
ANBY09	1.5	0.7
ANBY10	8	0.1
ANBY11	0.7	1.4
ANBY12	0.5	2
ANBY13	1	0.5
ANBY14	0.8	1.3

It has long been known that the effects of local geology on ground shaking represent an important factor in earthquake engineering. In particular, soft sedimentary cover could strongly amplify the seismic motion. The frequency band affected by such effects depends on the thickness and on the velocity of the sedimentary layers. When amplifications occur at frequencies close to the fundamental frequency of vibration of the buildings greater damages can be expected.

During an earthquake buildings oscillate, but not all buildings respond to an earthquake equally. If the frequency of oscillation of the ground is close to the natural frequency of the building, resonance (high amplitude continued oscillation) may cause severe damage. In the analysis of the H/V data attention was paid to past research where most examples reported in the literature indicate clear peaked H/V curve for soft soils and almost flat curves for rock sites. When the H/V peak is clear, then the site under study presents a large velocity contrast at some depth, and is very likely to amplify the ground motion.

Based on the H/V curves spectral ratios data in Annotto Bay there are sections in the town (see figure 5) that shows clear single frequency patterns with high amplification (Sites ANBY6, ANBY9, ANBY11, ANBY12, ANBY13, ANBY14) indicating characteristics of thick soil layer. These areas are expected to show high amplification in a major earthquake. Based on the period pattern in these communities there should no serious issues with resonance as the ratio between the resonance effect (0.1 sec/single storey) does not exist. Most buildings in the community are within 1-2 stories and the period pattern falls above these ratios so the issue of resonance is not a critical factor.
For future development, attention must be paid to the height of structures and the fundamental period of the different areas of the community as stability of buildings bear clear correlation with resonance.

3.4.3.2 RVS Analysis

Annotto Bay consists of nine building types namely: Reinforced concrete, Wood, W/Concrete, URM (Brick), Nog, Nog & Concrete, Wood/Nog/Concrete, Wattle and Daub and metal containers modified to serve mainly as small commercial buildings, see figure 8.

Of the approximately 1632 buildings in Annotto Bay, 1498 buildings were assessed. The three main building types include; Reinforced Concrete (970), Wood (443) and W/Concrete (69). These structures represented 64.8%, 29.6% and 4.6% percentage concentration respectively, while the remaining six building types had less than 0.1% concentration except for containers that had a concentration of 0.6% (See Figure 19 & 20).

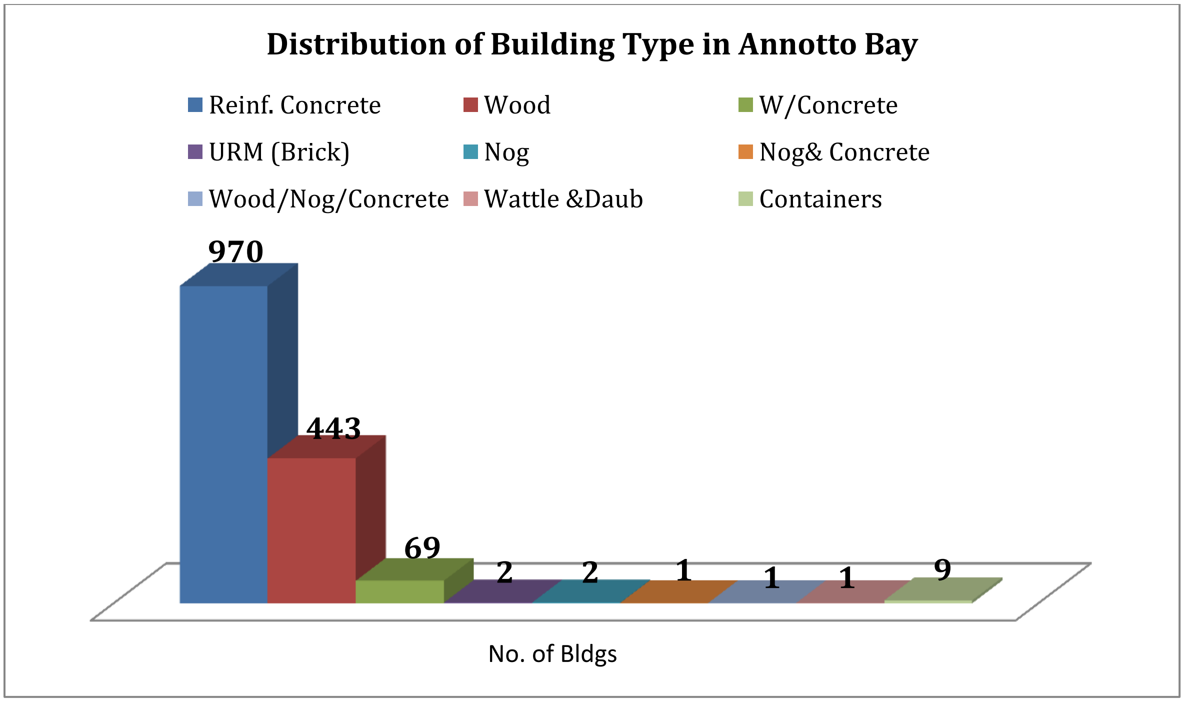


Figure 21: Number of and Building Type in Study Area.

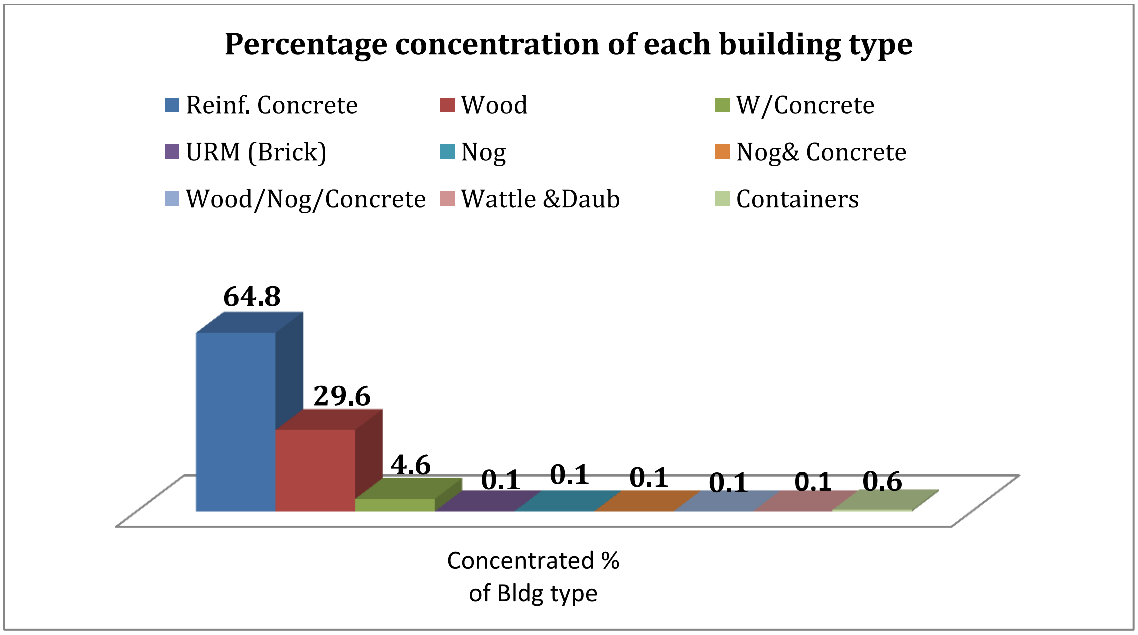


Figure 22: Percentage of Building Type in Study Area

Each variation of the specific building class is scored based on the applicable RVS modifiers. A post benchmark modifier was applied to those buildings that were built in the post 1983 when engineers applied regional building codes. However, in cases where this criterion was met but buildings were of poor quality (indicated by poor construction method or structural defects that would imply that adequate engineering consideration was absent). Hence, the final score produced by these structures would best reflect the average performance expected for these types of structures.

Reinforced masonry buildings with rigid floor and roof diaphragms (RM1)

Based on the RVS guidelines, the RM1 structures are given a basic score of 3.6 and the final score is calculated based on the applicable modifiers. These RM1 structures exist on soil types C-E, and the corresponding final scores were calculated for each of the structures found in each area based on soil type. See Tables 10-12 for the results of structural performance of RM1 buildings based on varied combinations of existing modifiers and soil type.

Table 10: Qualitative assessment of RM1 buildings based on applicable modifiers and resulting final scores on Soil Type C.

Base score of Building / Modifier Scores	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
Vertical irregularity	-2.0	-2.0	-	-	-2.0	-2.0	-	-	-
Plan irregularity	-0.5	-0.5	-	-0.5	-0.5	-	-	-	-
Soil type C	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8
Post benchmark	2.0	-	2.0	2.0	-	2.0	-	-	-
Pre-code	-	-	-	-	-0.4	-	-0.4	-	-0.4
Final Score	**2.3**	**0.3**	**4.8**	**4.3**	**-0.1**	**2.8**	**2.4**	**2.8**	**2.4**

Table 11: Qualitative assessment RM1 buildings based on applicable modifiers and resulting final scores on Soil Type D.

Base score of Building / **Modifier Scores**	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
Vertical irregularity	-2.0	-2.0	-	-	-2.0	-	-	-
Plan irregularity	-0.5	-0.5	-	-0.5	-0.5	-	-	-
Soil type D	-1.2	-1.2	-1.2	-1.2	-1.2	-1.2	-1.2	-1.2
Post benchmark	2.0	-	2.0	2.0	-	-	-	-
Pre-code	-	-	-		-0.4	-0.4	-	-0.4
Final Score	**1.9**	**-0.1**	**4.4**	**3.9**	**-0.5**	**2.0**	**2.4**	**2.0**

Table 12: Qualitative assessment of RM1 buildings based on applicable modifiers and resulting scores on Soil Type E.

Base score of Buildings / **Modifier Scores**	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
Vertical irregularity	-2.0	-2.0	-	-	-2.0	-	-	-
Plan irregularity	-0.5	-0.5	-	-0.5	-0.5	-	-	-
Soil type E	**-1.6**	**-1.6**	**-1.6**	**-1.6**	**-1.6**	**-1.6**	**-1.6**	**-1.6**
Post benchmark	2.0	-	2.0	2.0	-		-	
Pre-code	-	-			-0.4	-0.4	-	-0.4
Final Score	**1.5**	**-0.5**	**4.0**	**3.6**	**-0.9**	**1.6**	**2.0**	**1.6**

Unreinforced Masonry bearing-wall buildings (URM)

Brick, Nog and Wattle and Daub structures are all considered as unreinforced masonry (URM) and as such the final score is calculated using the URM category. It should also be noted that post benchmark doesn't apply to URM structures when using the moderate seismicity form. The basic score applied to URM structures is a score of 3.4, and as illustrated in the previous description of RM1 structures. Applicable modifiers are added to the score based on the variation of the building designs and soil types found in the specific area. Tables 13 - 15, illustrate the final scores produced by these structures for combination of modifiers that were applied for structures in the area.

Table 13: Qualitative assessment of URM buildings based on applicable modifiers and resulting final scores on Soil Type C.

Base score of Building / Modifier Scores	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
Vertical irregularity	-1.5	-1.5	-	-	-1.5	-	-	-
Plan irregularity	-0.5	-0.5	-	-0.5	-0.5	-	-	-
Soil type C	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4
Post benchmark	2.0	-	2.0	2.0	-	-	-	-
Pre-code	-	-	-	-	-0.4	-0.4	-	-0.4
Final Score	**3.0**	**1.0**	**5.0**	**4.5**	**0.6**	**2.6**	**3.0**	**2.6**

Table 14: Qualitative assessment URM building based on applicable modifiers and resulting final scores on Soil Type D.

Base score of Building / Modifier Scores	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
Vertical irregularity	-1.5	-1.5	-	-	-1.5	-	-	-
Plan irregularity	-0.5	-0.5	-	-0.5	-0.5	-	-	-
Soil type D	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8
Post benchmark	2.0	-	2.0	2.0	-	-	-	-
Pre-code	-	-	-	-	-0.4	-0.4	-	-0.4
Final Score	**2.6**	**0.6**	**4.8**	**4.1**	**0.2**	**2.2**	**2.6**	**2.2**

Table 15: Qualitative assessment URM building based on applicable modifiers and resulting final scores on Soil Type E.

Base score of Building / Modifiers Score	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
Vertical irregularity	-1.5	-1.5	-	-	-1.5	-	-	-
Plan irregularity	-0.5	-0.5	-	-0.5	-0.5	-	-	-
Soil type E	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6
Post benchmark	2.0	-	2.0	2.0	-	-	-	-
Pre-code	-	-	-	--	-0.4	-0.4	-	-0.4
Final Score	**1.8**	**-0.2**	**3.8**	**3.3**	**-0.6**	**1.4**	**1.8**	**1.4**

Steel moment - resisting frame buildings (S1)

S1 structures also included the metal containers that were modified primarily for use as small commercial buildings. Based on the RVS guideline S1 structures are given a basic score of 3.6. These structures were only found on soil type E and as such the appropriate modifiers were applied based on this soil type. Table 16 shows the resulting final score for these structures based on varied combination of modifiers that existed in the area.

Table 16: Qualitative assessment of S1 buildings based on applicable modifiers and resulting final scores on Soil Type E.

Base score of Building / Modifiers Score	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
Vertical irregularity	-1.5	-1.5	-	-	-1.5	-	-	-
Plan irregularity	-0.5	-0.5	-	-0.5	-0.5	-	-	-
Soil type E	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8
Post benchmark	2.0	-	2.0	2.0	-	-	-	-
Pre-code	-	-	-	-	-0.4	-0.4	-	-0.4
Final Score	**2.8**	**0.8**	**4.8**	**4.3**	**0.4**	**2.4**	**2.8**	**2.4**

Concrete moment-resisting frame buildings (C1)

C1 structures are given a basic score of 3.0 and similarly the applicable modifiers are added to the basic score as dictated by design and soil type. These structures were only found on soil type E and the resultant final scores for the varied combination of modifiers are illustrated in Table 17.

Table 17: Qualitative assessment of C1 buildings based on applicable modifiers and resulting scores on Soil Type E.

Base score of Building / Modifiers Score	3.0	3.0	3.0	3.0	3.0
Vertical irregularity	-2.0	-2.0	-	-	-
Plan irregularity	-0.5	-	-0.5	-	-
Soil type E	-1.6	-1.6	-1.6	-1.6	-1.6
Post benchmark	-	-	-	-	
Pre-code	-	-	-	-	-0.4
Final Score	**-1.1**	**- 0.6**	**0.9**	**1.4**	**2.2**

Light wood-frame residential and commercial buildings ≤ 5,000 square feet (W1)

W1 structures are generally very good seismic performers and has a basic score of 5.2; however, a large number of the structures in the study area were not properly constructed (make-shift plywood) and as such adjustments were made to factor in the likely reduced seismic performance. Usually a pre-code would normally be applied to adjust this shortcoming; however, FEMA does not provide a pre-code modifier for W1 structures. Therefore, the basic score for W2 was used to calculate the final score of W1 structures that 7fits the aforementioned scenario. W1 structures were found on all three soil types (C, D and E). As W1 structures are generally very good seismic performers only the combined modifiers scenarios existing on soil type E (worst case scenario) are illustrated in Table 18 below.

Table 18: Qualitative assessment of W1 building based on applicable modifiers and resulting final scores of buildings on soil type E.

Base score of Building / Modifiers Score	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
Vertical irregularity	-3.5	-3.5	-	-3.5	-	-3.5	-	-	-
Plan irregularity	-0.5	-0.5	-	-	-0.5	-0.5	-	-	-
Soil type E	-1.2	-1.2	-1.2	-1.2	-1.2	-1.2	-1.2	-1.2	-1.2
Post benchmark	1.6	-	1.6	1.6	1.6	-	-	-	-
Pre-code	-	-	-	-	-	-0.4	-0.4	-	-0.4
Final Score	**1.2**	**-0.4**	**5.2**	**1.7**	**4.7**	**-0.8**	**3.2**	**3.6**	**3.2**

4.0 VULNERABILITY ASSESSMENT

4.1 Introduction

This section of the risk assessment will evaluate the elements at risk to the various hazards in Annotto Bay namely, Earthquakes, Storm surge and flooding. Elements at risk defined in this assessment are housing, critical facilities (schools, hospital, health centre, fire station and police station), population and agriculture which are exposed to the aforementioned hazards. The vulnerability assessment will focus on:

- Physical vulnerability – the potential for physical impact on the built environment and population.
- Social vulnerability - provides an estimation of vulnerable populations in hazards areas
- Economic vulnerability - the potential impacts of hazards on economic assets and processes;

The vulnerability and risk assessment has been carried out for various severities of storm surges and the 2001 flood event. For seismic vulnerability, the analysis will indicate which structure (s) are deemed seismically hazardous and required detailed engineering assessment.

4.2 Methodology

The vulnerability and risk assessment is conducted using two (2) distinct methodologies:

1. Utilizing a Geographic information System (GIS) based analysis
2. Applying a statistical/probabilistic risk assessment

4.2.1 GIS-Based Analysis

The objective of the GIS analysis is to determine patterns and relationships using several vulnerability datasets. These data, when overlain on the hazard maps provide the estimated vulnerability of people, number of buildings and critical facilities as well as economic activities to the identified hazards for Annotto Bay. By understanding the type and number of assets that exist and where they are located in relation to known hazards areas, the relative risk and vulnerability for such assets can be assessed. An overview of the methodology for the risk assessment is presented in Figure 22.

Elements at Risk Inventory

An element of risk database was generated focusing on buildings and population. To facilitate this level of analysis detailed building inventory mapping was conducted with mobile GIS using Global Positioning System (GPS). The GPS device creates an accurate positional reference of the buildings in the community. Each building is represented as a point (see Figure) and a total of 1632 buildings were collected to include houses, critical facilities, financial institutions, offices and other buildings in the community. The data was collected over a 5 day period with a three (3) member team from the ODPEM along with six (6) community representatives.

A data dictionary was created to collect the following attributes for each building:

i. Land use and building type – categories used were defined by NEPA (See Annex C) for classification)
ii. Material of construction – as defined by STATIN
iii. Number of floors – direct observation
iv. Size/building footprint of critical facilities- direct measurement of the square foot of the 13 critical facilities.
v. Replacement value of critical facilities- the square footage of the buildings is used to estimate the replacement value of critical facilities. This was calculated by a quantity surveyor.
vi. Content value- estimation of value of contents of building such as equipment and furniture.

Other attributes such as the height of finished floor level and building footprint of houses were averaged based on random sampling of buildings in the community. The replacement value reflects present day cost of labour and materials to construct a building of similar size, type and quality.

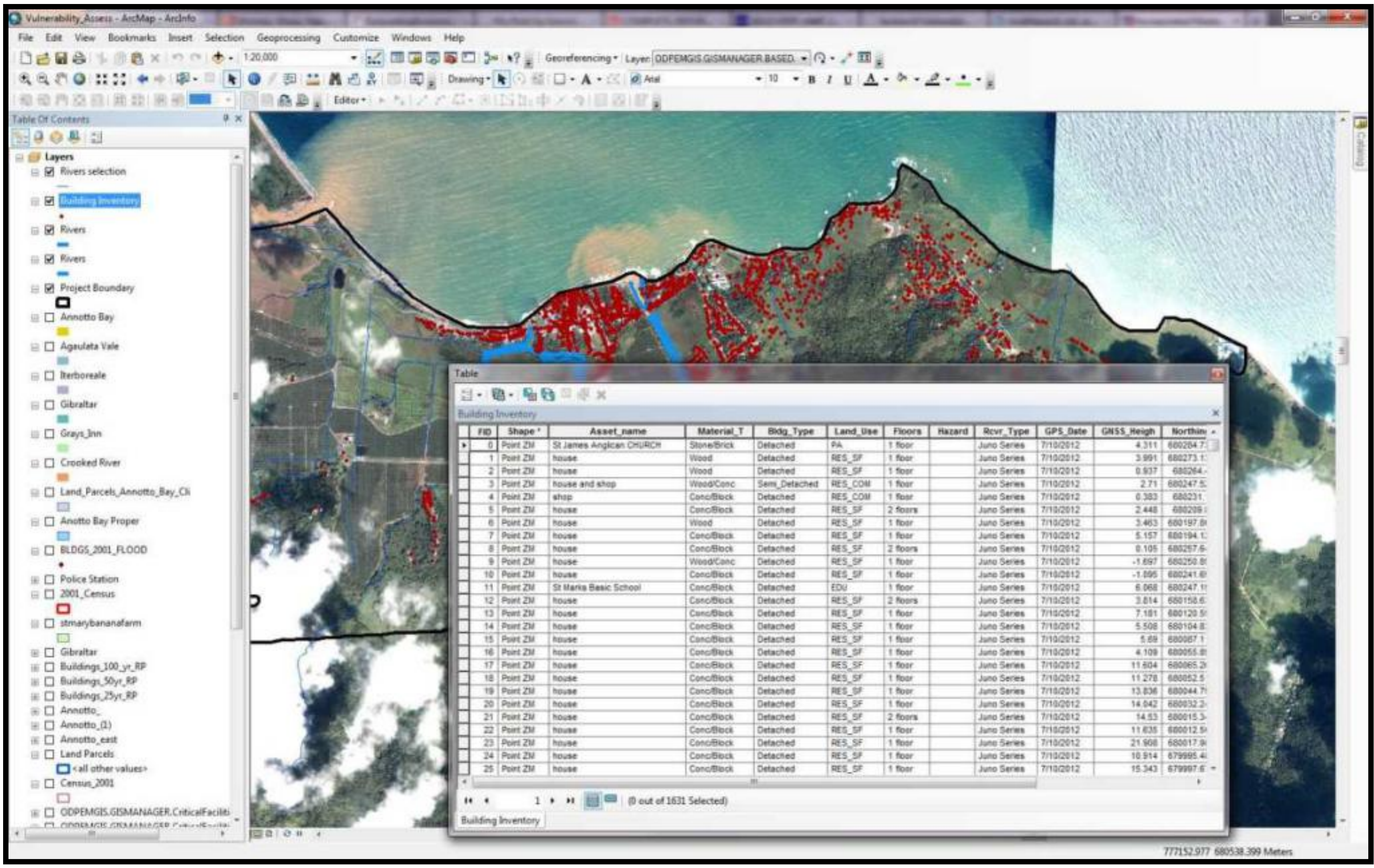


Figure 23: Illustration of Building Inventory with Attributes in GIS Environment

Physical and Economic Vulnerability

For physical and economic vulnerability assessment, the inventoried assets were overlaid with the hazard maps for storm surge, 2001 flood event in Annotto Bay and the seismic Iso-period determined from the fundamental frequency ratio for Annotto Bay. The overlapping of areas of the hazard maps and building inventory data allow for identification of different elements at risk. The results of the analysis provided an estimate of the number of people, buildings, and critical facilities, as well as the value of buildings, determined to be potentially at risk to those hazards with delineable geographic hazard boundaries.

To calculate the expected flood depth (m) associated with each storm surge return period or scenario, individual raster maps of each of the storm surge levels or return period was created. The existing topographic data or land surface raster was then subtracted from the individual storm surge raster data which was created based on return period using the raster calculator tool in ArcGIS. The extract value to point function in arc toolbox was used to create a field in the building inventory attribute table for flood depths at that location.

Population Vulnerability

Using the 2001 Census population data, vulnerable population in hazard areas was estimated by intersecting Enumeration Districts with hazard areas were used to determine exposed population counts.

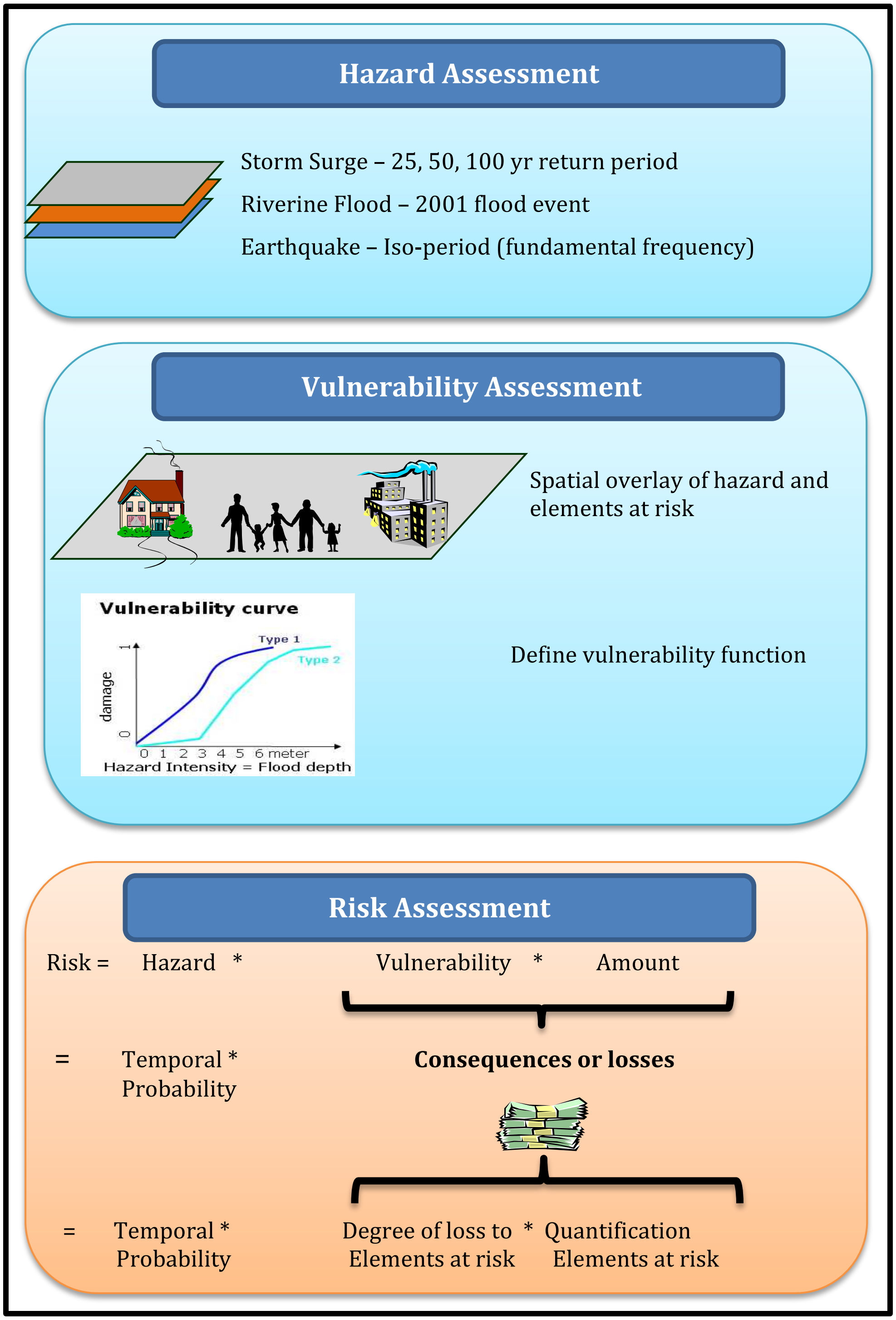


Figure 24: A summation of the Risk Assessment Methodology

4.3 Vulnerability Assessment

An inventory of geo-referenced assets in Annotto Bay was compiled in order to identify and characterize the buildings potentially at risk to the identified hazards. Figure 24 illustrates the spatial distribution of building inventory in Annotto which consist of one thousand six hundred and thirty two (1632) buildings, most of which are concentrated in and around the town centre.

4.3.1 Description of Building Inventory

As detailed in Chapter 3, there are seven (7) main types of buildings in Annotto Bay, however the predominant material of construction are reinforced concrete, wood and combination of wood and concrete as shown in Figure 19.

Table 19 shows the land use and the number of structures within each category. Residential represents the highest number of structures with 1349; however a detailed breakdown of this category indicated that informal residential unit account for 35% of the total. Both single and multi-family houses account for 42% of the building stock in Annotto Bay. This is followed by lands used for commercial purposes which totaled 145 buildings. It must be noted that there are quite a few vacant buildings in the community which amounted to 76.

Table 19: Number of buildings by Land Use Category

# of Parcels	Land Use	# of Buildings
1226	**Residential**	
	- *Informal*	566
	- *Single Family*	562
	- *Multi-family*	131
	- *Commercial*	42
	- *Informal commercial*	9
	- *Educational*	1
	- *Public Assembly*	1

# of Parcels	Land Use	# of Buildings
	TOTAL RESIDENTIAL	**1349**
	Commercial	145
	Light Industry	12
	Public Assembly	19
	Educational	8
	Office	10
	Public Buildings	4
	Recreational	2
	TOTAL	**1632**

Based on the building inventory, 1527 are constructed with one (1) floor, 98 buildings with two (2) floors and 7 buildings with three (3) floors. The average height of finished floor level for all buildings from surface is one (1) foot 6 inches (1.5 ft). The height of the finished floor from the surface is defined as the height difference between the surrounding surface with finished floor level.

There is a relationship between the number of floors and the different areas in Annotto Bay. The middle and upper middle income areas of Iterboreale and Gibraltar contain 69% of the buildings with two (2) floors whilst the remainder is concentrated in the town centre and are used for commercial or residential commercial. The same is also true that the housing stock in the two aforementioned areas is of a better quality than those in the Annotto Bay zone.

Table 20: Material Type and Number of Floors

Material Type	Number of Floors			Total
	One floor	Two floors	Three floor	
Reinforced Concrete	853	94	7	954
Wood	517	2		519
Wood/concrete	131	2		133
Stone Brick/URM	6			6
Nog	4			4
Wattle/Daub	1			1
Zinc/Container	15			15
Total	**1527**	**98**	**7**	**1632**

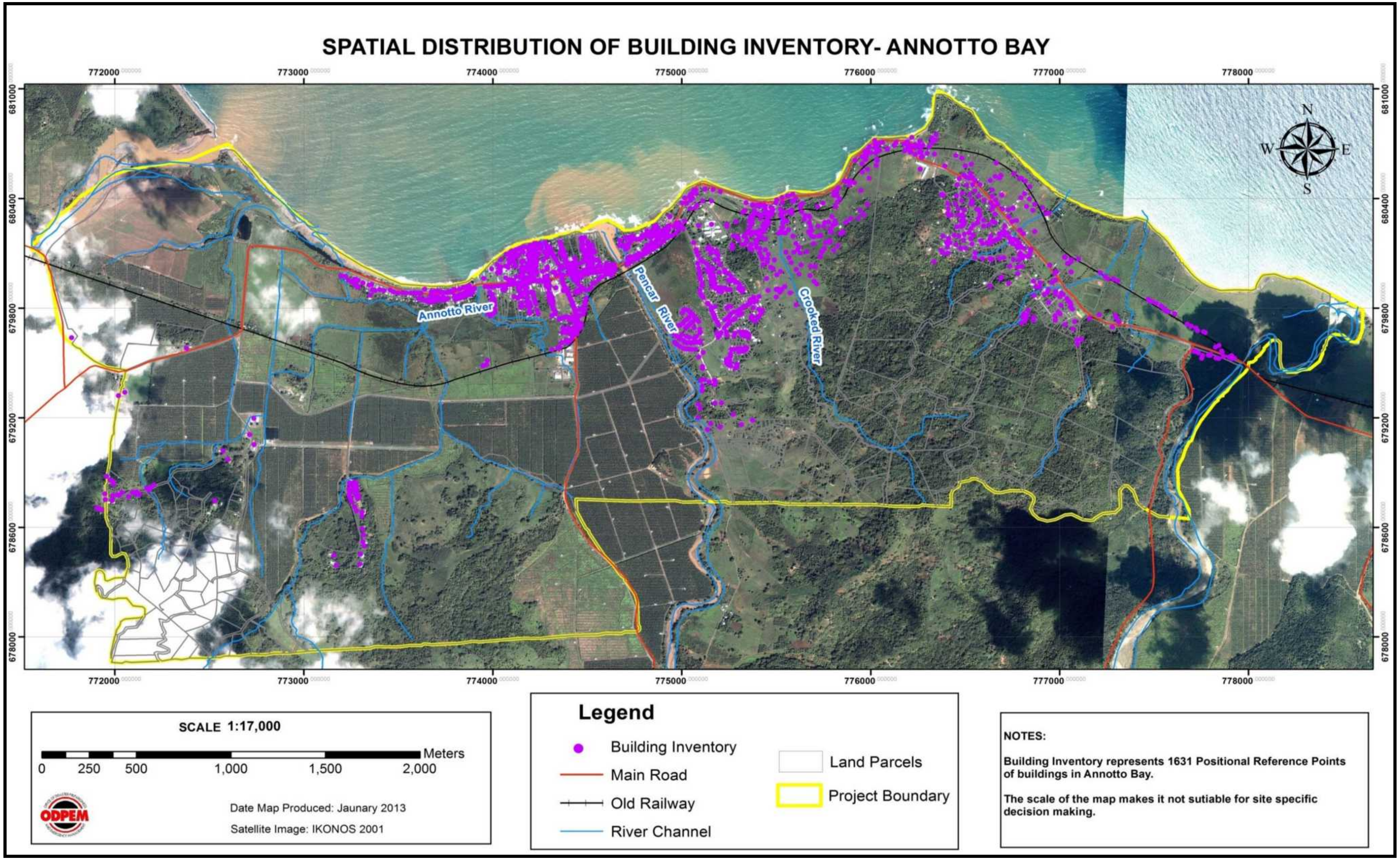


Figure 25: Spatial Distribution of Buildings in the Study Area

4.4 Storm Surge Vulnerability Assessment

4.4.1 Physical Vulnerability

There are a total of 621 buildings for determined to be vulnerable to the effects of storm surge as these structures are located within the 100 years return period. Figure 25 shows the spatial distribution of buildings that are exposed to storm surge for the 25- 100 year return period scenario. For properties on the seafront they will be more exposed to wave action as well as flooding and will be quickly destroyed. That is, the sheer hydraulic force of the waves will damage the property. On the other hand, structures further inland will be exposed to what is referred to as "resting" water damage. The entire town centre, a number of government facilities and institutions namely the Annotto Bay Police Station, Fire Station, Annotto Bay Health Centre, Annotto Bay All Age, Inland Revenue Department and Court house among others are vulnerable to flooding. Table 21 depicts the proportion of buildings in Annotto Bay that are located in the 25, 50 and 100 storm surge hazard areas.

Table 21: Proportion of Buildings that are located in storm surge zone

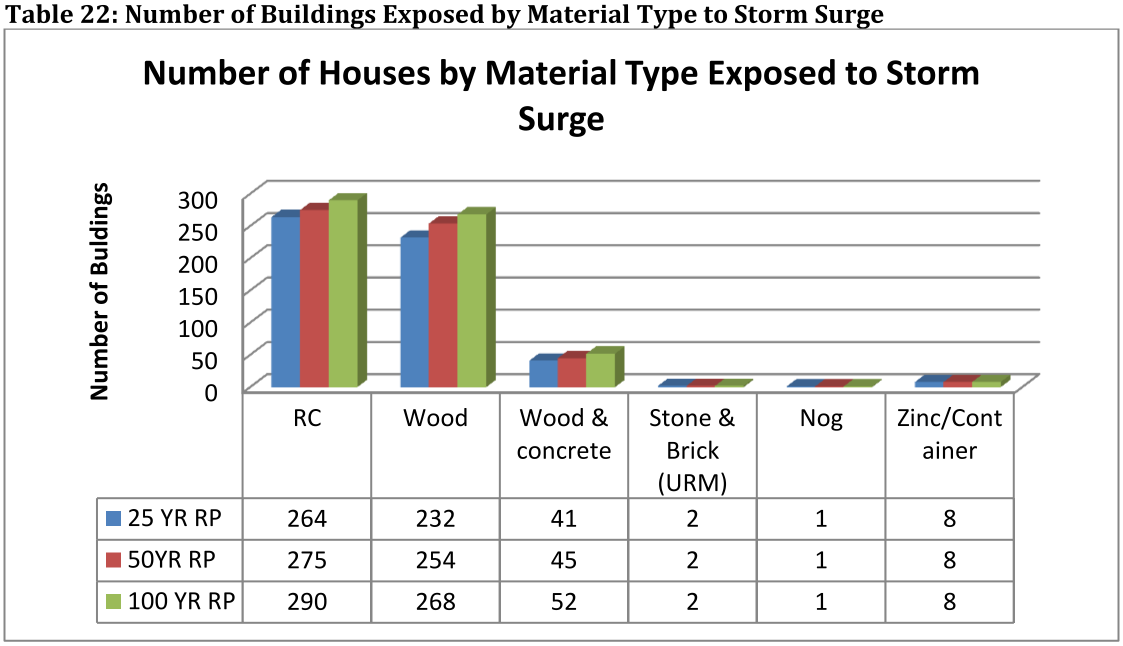
Land Use	Total Assets	Hazard: Storm Surge					
		Number of Structures					
		25 YR RP	% in Hazard Area	50 YR RP	% in Hazard area	100 YR RP	% in Hazard Area
Residential	1349	411	31%	438	33%	468	35%
Commercial	146	93	64%	103	71%	105	72%
Industrial Light	12	7	58%	7	58%	7	58%
Educational	8	4	50%	4	50%	4	50%
Office	10	8	80%	8	80%	9	90%
Public Assembly	19	13	68%	13	68%	14	74%
Public Buildings	6	5	83%	5	83%	5	75%
Recreational	2	0		0		0	
Derelict Building	1	1	100%	1	100%	1	100%
Vacant Buildings	76	8	11%	8	11%	10	13%

Land Use	Total Assets	Hazard: Storm Surge					
		Number of Structures					
Total	**1632**	**548**	**34%**	**585**	**36%**	**621**	**38%**

4.4.2 Housing Sector

The housing sector would be the worst affected in terms of the number of buildings in the impact zone. A total of 468 houses are located in the 100 year storm surge return period scenario, representing 35% of all residential units are exposed to flooding with maximum flood heights of 1.9m, 2.1m and 2.3m for the 25 year, 50 year and 100 year storm surge return periods. Furthermore of the 468 houses, 265 (56%) are informal houses, a number of which are located on the seafront with a distance of approximately 17.5 m from the high water mark which are expected to suffer the most damage. Many of these structures are poorly constructed of wood with concrete or wood flooring and will not be able to withstand the force of storm surges. Table 22 shows the distribution of houses by building type exposed to storm surge for the different return periods.

Table 22: Number of Buildings Exposed by Material Type to Storm Surge



	RC	Wood	Wood & concrete	Stone & Brick (URM)	Nog	Zinc/Container
25 YR RP	264	232	41	2	1	8
50YR RP	275	254	45	2	1	8
100 YR RP	290	268	52	2	1	8

The recent passage of Hurricane Sandy, 2012 (see Figure 26) further reinforced the physical vulnerability of structures in the community, particularly seafront properties.

Even though the hurricane was just category 1, seven (7) houses were completely destroyed whilst another13 structures sustained major damage.

Plate 1: Resident showing Flood Height

a. Resident showing height of flood water - Purcell Lane

Plate 2: Debris Along Shoreline after Hurricane Sandy

b. Shoreline after the passage Hurricane Sandy

Plate 3: A house that was totally destroyed by hurricane

c. Purple outline showing only foundation of where a house once stood

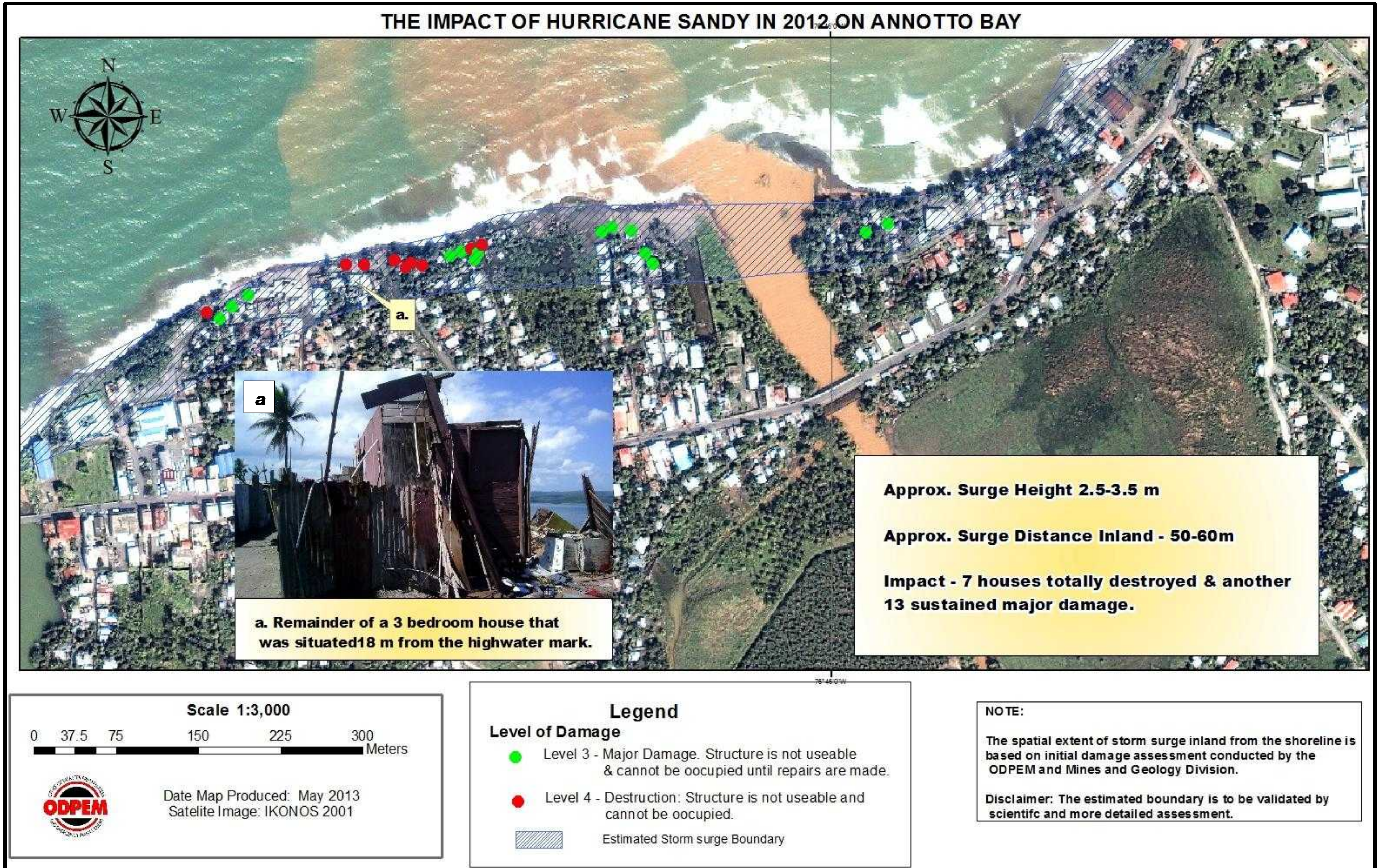


Figure 26: Physical Impact of Hurricane Sandy in Annotto Bay, 2012

4.4.3 Critical Facilities

Critical facilities are essential to the health and welfare of the community and are especially important to the response and recovery efforts following hazard events. There are thirteen (13) critical facilities in Annotto Bay, 8 of which are educational institutions, a fire station, a police station, Inland Revenue, health centre and hospital. Eight (8) of the thirteen inventoried critical facilities are vulnerable to the effects of storm surge as they are located within the 25 to 100 year storm surge scenario and . These facilities are:

1. Annotto Bay Health Centre
2. Annotto Bay Fire Station
3. Annotto Bay Police Station
4. Inland Revenue Department and Court
5. Annotto Bay All Age School
6. Dorcas Basic School
7. ABC Smart Kids Learning Centre
8. Gospel Chapel Preparatory

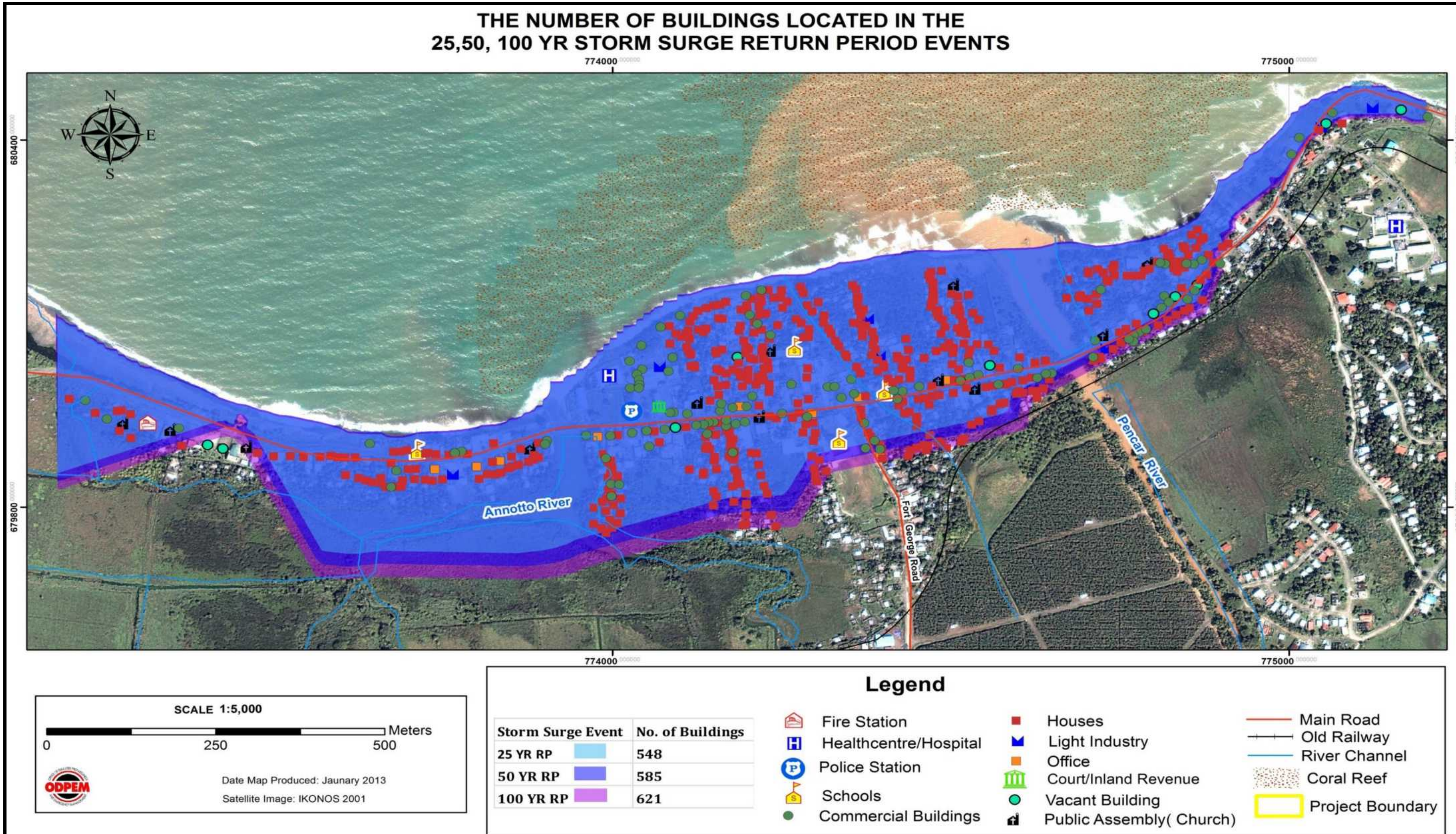
From the list above it can be seen that the first responders, namely fire and police stations are vulnerable which has implications. That is, if both facilities are affected it may limit their ability to respond effectively to emergencies. The Annotto Bay Health Centre and Gospel Chapel Preparatory School are directly located on the seafront and will not only be exposed to wave action but flooding as well. Based on the location of the named facilities except for ABC Learning and Annotto Bay All Age are situated in the storm surge boundary associated with Hurricane Allen in 1980 which underscores the vulnerability of these facilities. According to Wilmot et al (1980), … "several *walls for example the area of the fire station was demolished*". Anecdotal evidence indicated that the town centre was inundated for approximately two (2) days.

Given that the average finished floor level of buildings in Annotto Bay is 1.5ft from the surface, many of these structures especially those closer to the shoreline would experience up to 5 feet of water in the 100 year storm surge event.

4.4.4 Infrastructure Vulnerability to Storm surge

The main corridor which links Annotto Bay to the parishes of St. Andrew and Portland is exposed to the effects of storm surges. Approximately 1.9 km of the highway is vulnerable to flooding associated with the 25- 100 year storm surge return period scenarios. Some areas of the highway are more exposed particularly in the vicinity of the old railway station and the section from the fire station to the Annotto Bay River Bridge. Of the total, 0.75 km (750m) of the highway in the vicinity of the old railway station is directly exposed to wave action and which according to Wilmot et al (1980) was flooded during Hurricane Allen. This is the only access road to the Annotto Bay High School which substitutes as the priority emergency shelter for the community.

The sewerage plant that serves the community is located outside the delineated 100 year storm surge boundary for the community. This sewer system serves the Annotto Bay Hospital. The water treatment plant in Iteroboreal is also located outside of the 100 year storm surge period.



Storm Surge Event	No. of Buildings
25 YR RP	548
50 YR RP	585
100 YR RP	621

Figure 27: Buildings that are exposed to Storm Surge Hazard

4.4.5 Social Vulnerability to Storm Surge

It is important to identify and assess the population in Annotto Bay that is potentially at risk to storm surges. Analysis for population exposure is based on total number of population and age distribution in the community with regards to economic activity (dependent and working age). The exposure of the population to storm surge is presented in Table 23 and Figure 26.

For the 100 year storm surge scenario, a total of 2010 persons representing 37 per cent of Annotto Bay's population of 5422 are vulnerable to coastal flooding. Most of these persons are located in the densely populated Enumeration Districts – 013, 015, 016 and 017. These EDs also have the highest concentration of informal houses in the community and some of these persons may require short term shelter after a storm surge event. Table shows the population that is exposed to each storm surge scenario.

Table 23: Number of persons exposed to storm surge hazard

Population (2001 Census)	Hazard: Storm Surge					
	Exposed Population					
	25 YR RP	**% in Hazard Area**	**50 YR RP**	**% in Hazard area**	**100 YR RP**	**% in Hazard Area**
5422	1848	34%	1929	35%	2010	37%

The analysis further reveals that approximately 1427 persons classified as being in the productive age cohort (15-64) is located in the 100 year zone while 583 persons are classified as dependent population (children under 15 and elderly 65 years and above) is exposed to the 25-100 storm surge return period. The dependent population is considered the most vulnerable in the community who requires special attention during an emergency.

The population in EDs SE013, SE016, and SE017 representing 1175 persons would have to be evacuated and sheltered because all three (3) EDs are completely within the 25-100 year storm surge hazard footprint.

Using the 2011 Census average of 3 persons per household in Annotto the estimated number of informal settlers is 795 persons that require special considerations during such events.

265 informal houses x 3 persons/HH

=

Approx. 795 squatter population located within 100 yr storm surge scenario.

The vulnerable population who are located within the storm surge flood boundary would need to evacuate when a hurricane threat becomes imminent. If however, as was the case during Hurricane Sandy, 2012, residents refused to evacuate because of concerns pertaining to the security of their property and personal belongings, could result in high injury rates or even death as these persons put themselves at risk. To cite an example, interviews with residents after the hurricane indicated that some persons narrowly escaped death as one of the occupants of the 7 houses that were destroyed by storm surges was trapped by the building when it collapsed. This cultural behaviour or perception is exacerbated by as noted above that approximately 750m of the highway in the vicinity of the old railway station is directly exposed to wave action. time delays when an evacuation order is given can prove to be detrimental to the residents. It must be noted however that evacuations are not mandatory.

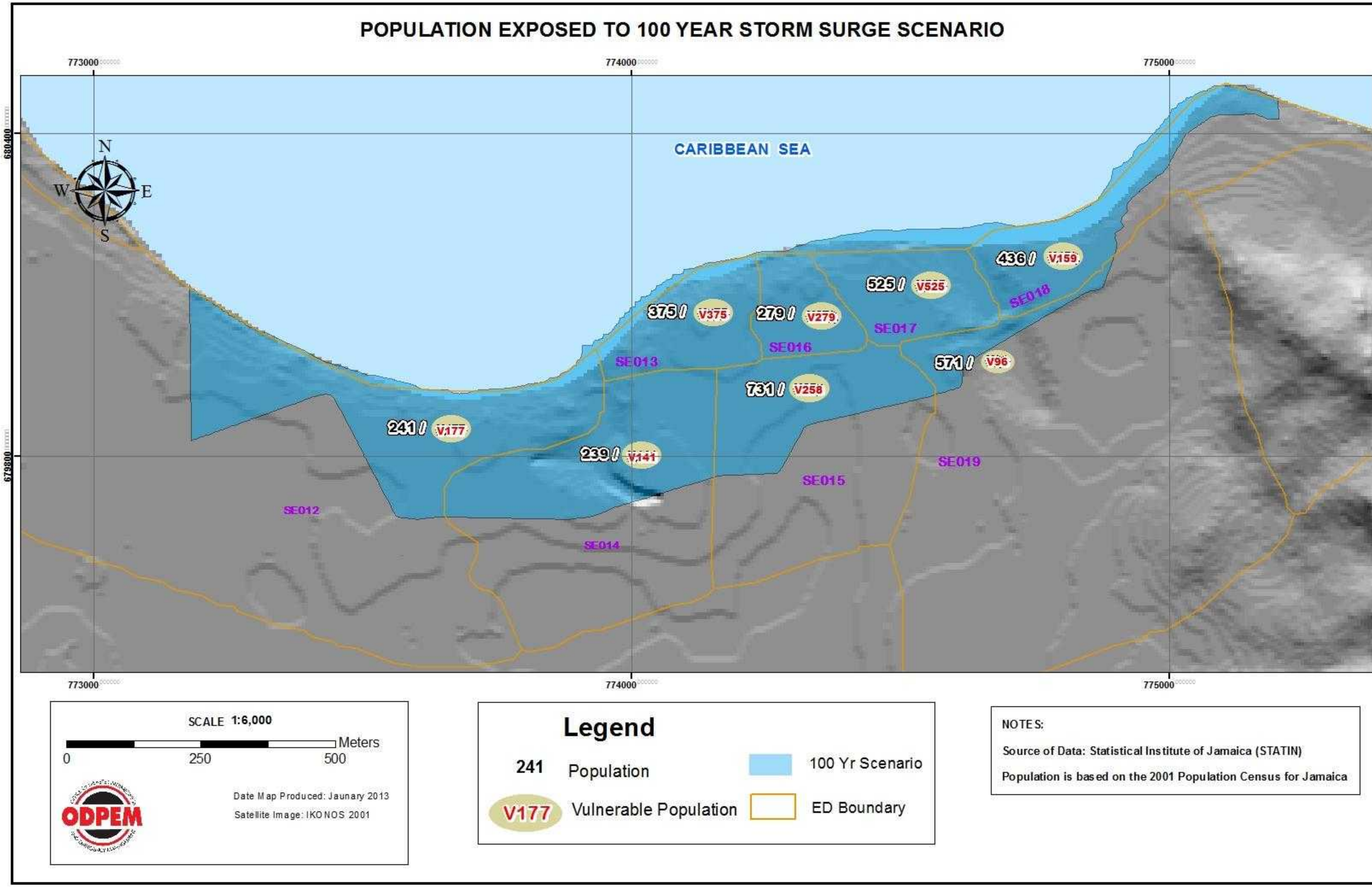


Figure 28: Number of Persons Exposed to Storm Surge Hazard

4.5 Riverine Flood Vulnerability Assessment

In the absence of the floodplain mapping scenarios for Annotto Bay, the 2001 flood event will be used to undertake a preliminary flood risk assessment to identify and analyze the elements at risk. When the floodplain scenarios are complete, this section of the report will be adjusted to reflect same.

4.5.1 Physical Vulnerability

A total of 729 structures were affected by flood waters with heights of 1-4 ft. Riverine flood is expected to affect a larger area compared to storm surges as the community is situated between the active floodplains of two rivers. Flood height in the town centre ranged from 1- 2 ft which affected commercial activities, government facilities and other buildings for at least one (1) day. Table 24 illustrates the number of structures and the percentage of structures located in the hazard area.

4.5.2 Housing Sector

Approximately 593 houses in Annotto Bay were affected by varying water depths. Areas adjacent to the river channels or basins experienced higher flood heights (see Figure 28). Flooding leads to damage and loss of household contents and impacts on the functionality of the household. The short duration flood event, however resulted in no significant physical damage to houses but temporary loss of use of contents. With the average finished floor level of 1.5 ft the maximum flood depth in the community was 2.5 feet but was confined to the agricultural lands belonging to St. Mary Banana Estate/Jamaica Producers. The depth of flooding relative to the finished floor level is the difference between the height of the water and height of the first floor or finished floor level of the structure. That is, all flood depths are relative to the elevation of the finished floor level. Houses in Cane Lane and sections of Fort George Road had flood depths of 1.5 ft whilst Dump and Cargill Lane 0.5 ft. In the case of town however, the depth of inundation was either 0.5 feet or at finished floor level.

Figure 27 illustrates the number of houses by material type that was affected by flooding. Approximately 60 per cent of the houses are informal, many of which are constructed of wooden material.

Table 24: Proportion of Buildings Affected by 2001 Flood Event

Hazard: Riverine Flood			
Land Use	**Total Assets**	**Number of Structures**	
		2001 Flood Event	**% in Hazard Area**
Residential	1347	591	44%
Commercial	146	109	75%
Agricultural	2	2	100%
Industrial Light	12	5	42%
Educational	8	4	50%
Office	10	10	100%
Public Assembly	19	13	68%
Public Buildings	6	5	83%
Recreational	2	0	0
Derelict Building	1	1	100%
Vacant Building	76	10	13%
Number of Buildings	**1632**	**749**	**46%**

Even though the Gibraltar Housing Scheme is located 35.3 meters from the Pencar River the houses were not affected because the area is of a higher elevation than surrounding areas.

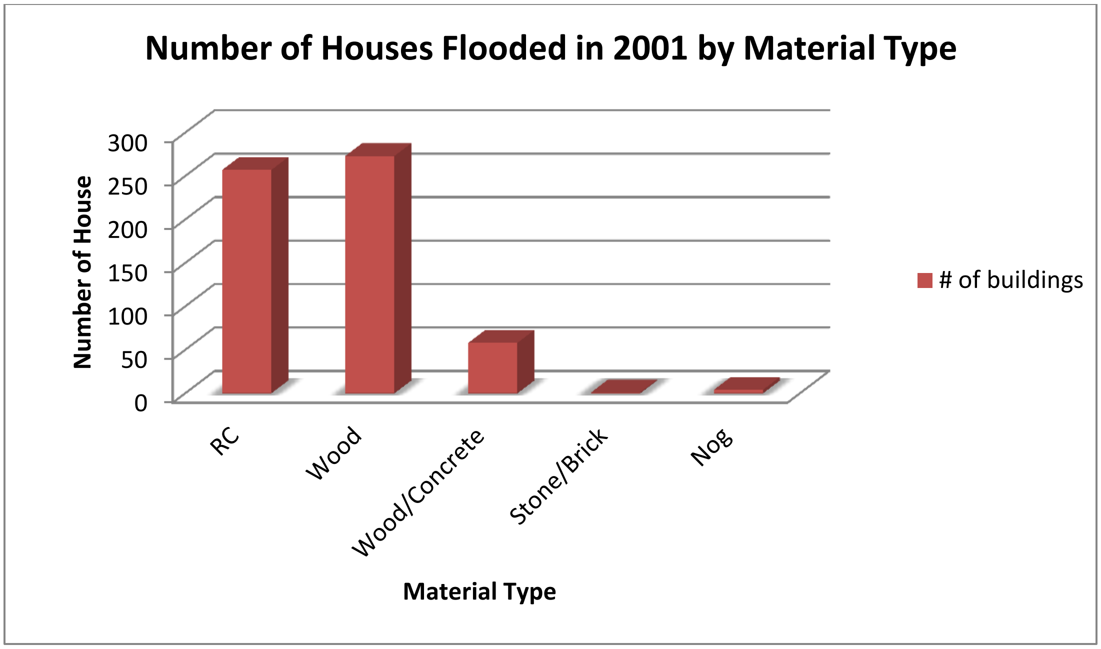


Figure 29: Number of Houses by Material Type Impacted by 2001 Flood Event

4.5.3 Critical Facilities

The same critical facilities that are exposed to storm surge hazard are also exposed to the riverine flooding and this is primarily due to their location, in the low lying areas of the community. The facilities were affected by varying flood depths from the 2001 event which significantly impacted the operations of these facilities (see below). Normal school activities, for instance, were interrupted for up to a week and the prisoners at the police station had to be relocated because the jail cells are situated on the first floor of the building which was flooded. Even though the Inland Revenue Department and Court are located on the second floor of building, access was and will continue to be an issue during instances of flooding in the town centre. These essential facilities in general were unable to function and serve the community effectively.

1. Annotto Bay Fire Station- 1.5 ft
2. Annotto Bay Police Station – at finished floor level
3. Inland Revenue Department & Court –at finished floor level
4. Annotto Bay Health Centre – at finished floor level
5. Annotto Bay All Age- 0.5 ft

6. Dorcas Basic School- 0.5 ft
7. ABC Smart Kids Learning Centre – 0.5 ft
8. Gospel Chapel Preparatory- 1.5 ft

4.5.4 Infrastructure Vulnerability to Riverine Flood

Similar to storm surge hazard, the highway is also exposed to riverine flooding as indicated in Figure 15 above. Approximately 3.5 km of the high way is exposed to riverine flood which will impede access through the town. Exposure of the road way to riverine flood extends from the Fire Station in the west to Gibraltar Housing Scheme Road to the east and will be a constant threat owing to location along the coast and on the floodplain of the Pencar and Annotto Rivers.

4.5.5 Social Vulnerability to Riverine Flood

The Enumeration Districts (EDs) SE013, SE014, SE015, SE016, SE017 and the built up areas of SE012 were completed inundated whilst only sections of EDs SE018, SE019 and SE 020 were impacted. The other EDs namely, SE021-SE023 were not affected because they are located outside the floodplain of the Pencar River as well as situated at a higher elevation. The 2001 flood event directly impacted 2740 people and indirectly the entire population of 5421 as access to the town and other services and amenities in the community was affected. Figure 28 depicts the EDs and corresponding population that were affected by flooding.

Table 25: Population Exposed to Riverine Flooding

Vulnerable Population	
Age Cohort	No. of persons
0-14	937
15-64	1607
65+	196
TOTAL	2740

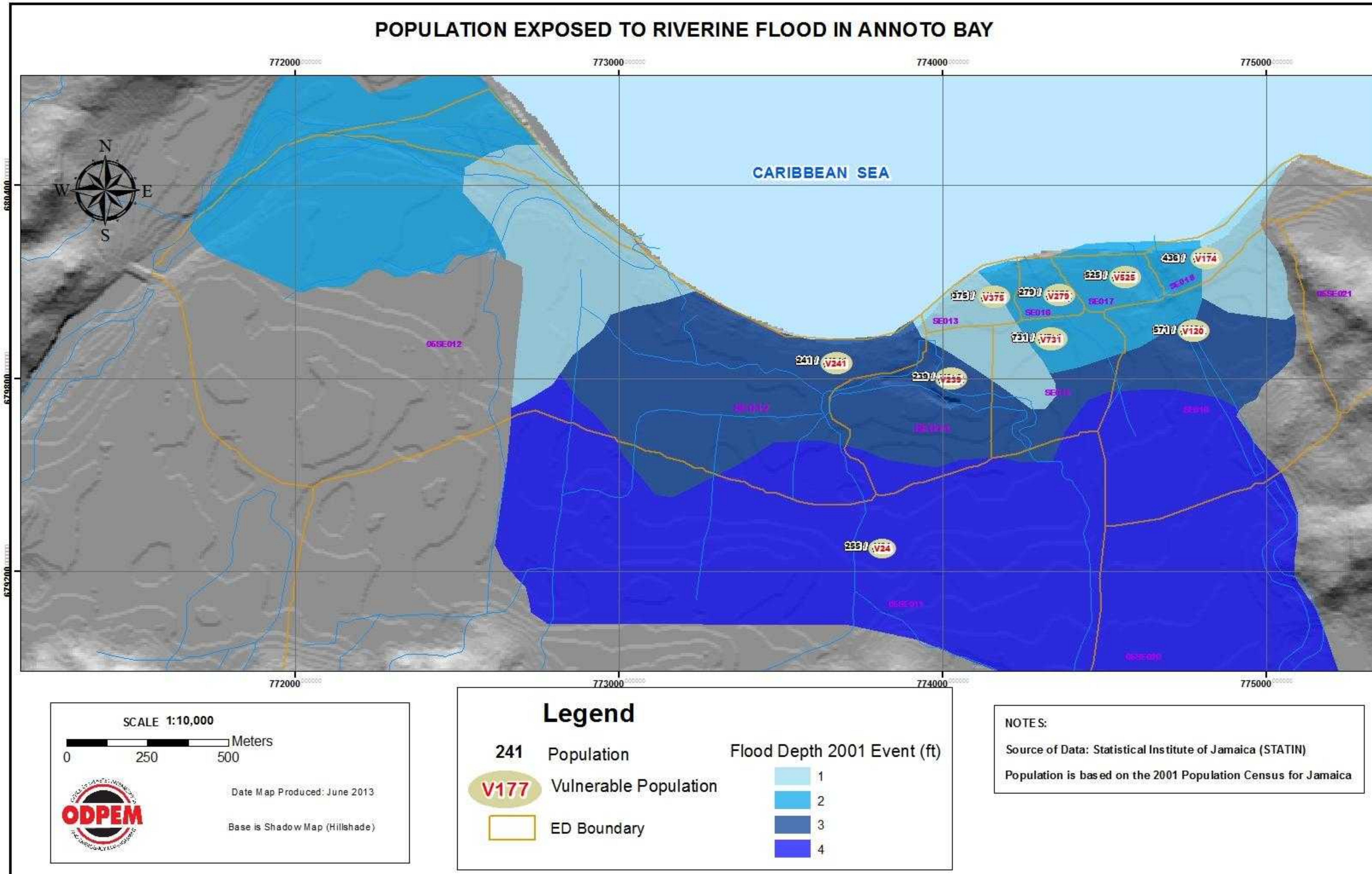


Figure 30: Exposed Population to Riverine Flood

4.6 Earthquake Vulnerability

4.6.1 RVS Performance

Iterboreale

The underlying soil type (See Chapter 3) in Iterboreale includes type C and D, and housed only three building types namely RM1, W1 and W/Concrete. Reinforced concrete structures in this area reflected moderate to good seismic performance producing final performance scores between 3.9 and 4.3. This district is a relatively young (most building in this area is less than 15 years) these structures were recently and properly constructed, thus the post code modifier was applied. However, in circumstances where vertical irregularities were present they reflected seismic vulnerability with reduced performance scores ranging from 1.9 and 2.3. On the other hand the W/Concrete structure existed only on soil type D and proved to be seismically vulnerable as it produced a final score of 2.0

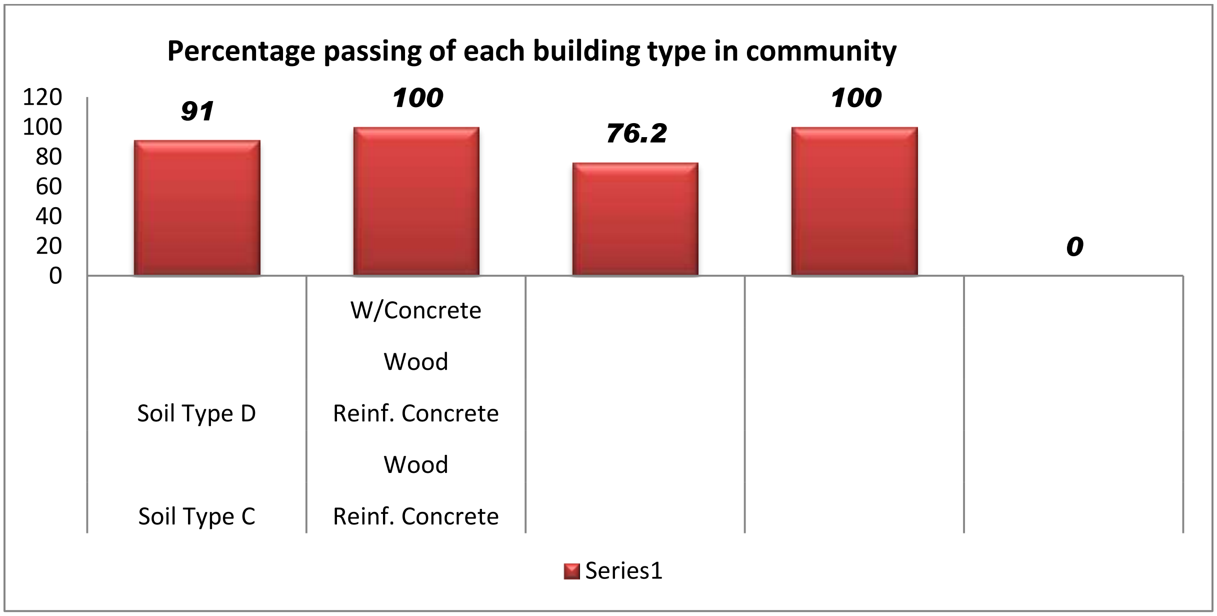


Figure 31: Bar Chart illustrating percentage passing of each building type in Iterboreale when modifiers are applied.

Additionally, wood structures found in this area reflected very sound performance in both soil type C and D producing final scores of at least 4.6 and 3.7 respectively (See Table 25).

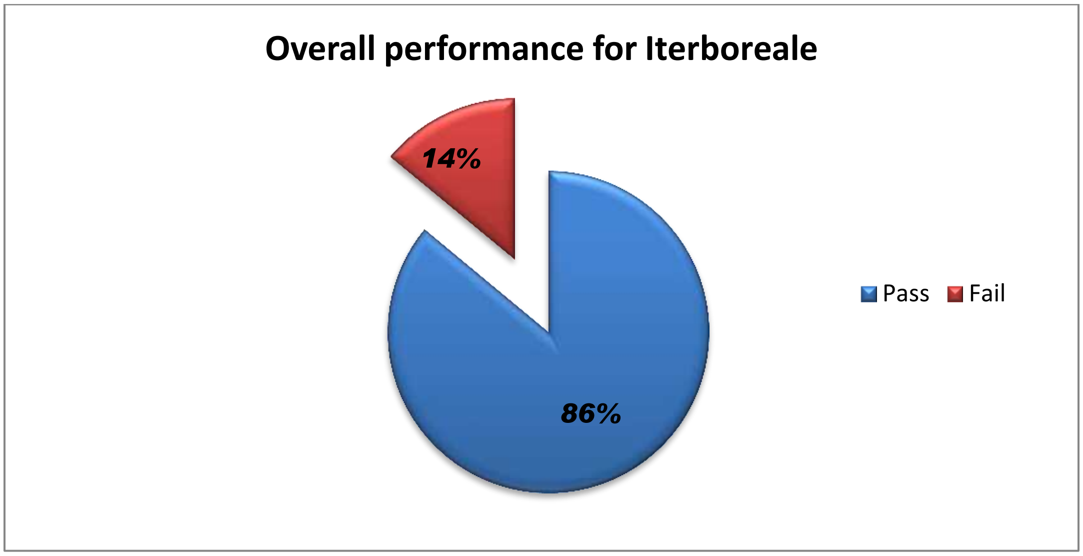


Figure 32: Pie Chart illustrating overall performance in the community of Iterboreale

As most of the structures found in this area were newly constructed reinforced concrete structures (92% concentration) and most of these were located on Soil type C, eighty six percent (86%) of the overall 197 structures passed (no detailed assessment is necessary) with the lowest final score being 3.7, while the minority fourteen percent (14%) failed (required detail assessment) with final scores ranging between 1.9 and 2.4. The major contributors to building failure in this community are as a result of vertical irregularity of buildings and the soil type D.

Table 26: Summary of the performance of structures in Iterboreale

Soil Type	No. of Buildings	Building Type	Plan Irregularity(%)	Vertical Irregularity (%)	Both Vertical & Plan Irregularity (%)	No Irregularity (%)	Properly Constructed (%)	Poorly Constructed (%)	Pass		Failed	
									%	Final scores 'S'	%	Final scores 'S'
C	119	Reinf. Concrete	41	0	9	50	100	0	91	≥ 4.3	9	2.3
	2	Wood	0	0	0	0	50	50	100	≥4.6	0	-
D	63	Reinf. Concrete	19	8	16	57	100	0	76	≥3.9	24	1.9 - 2.4
	12	Wood	25	0	0	75	8	92	100	≥3.7	0	-
	1	W/Concrete	0	0	0	0	0	100	0	-	100	2.0
Overall Performance	197								86	≥3.7	14	1.9 - 2.3

The area bordered by Iterboreale to the east and the Hospital road to the west

The community located to the west of Iterboreale and east of the Hospital road sits only on soil type D. Wood structures reflected very good seismic performance (S-score 3.7- 4.6) while reinforced concrete reflected fairly good performance (S-score 3.9 - 4.4). This is result of the fact that most of these structures did not have any irregularities. On the other hand W/concrete structures proved very detrimental as all (4) existing buildings reflected seismic vulnerability as they were improperly constructed (vertical and plan irregularities) and also mostly old structures (S-score 2.0).

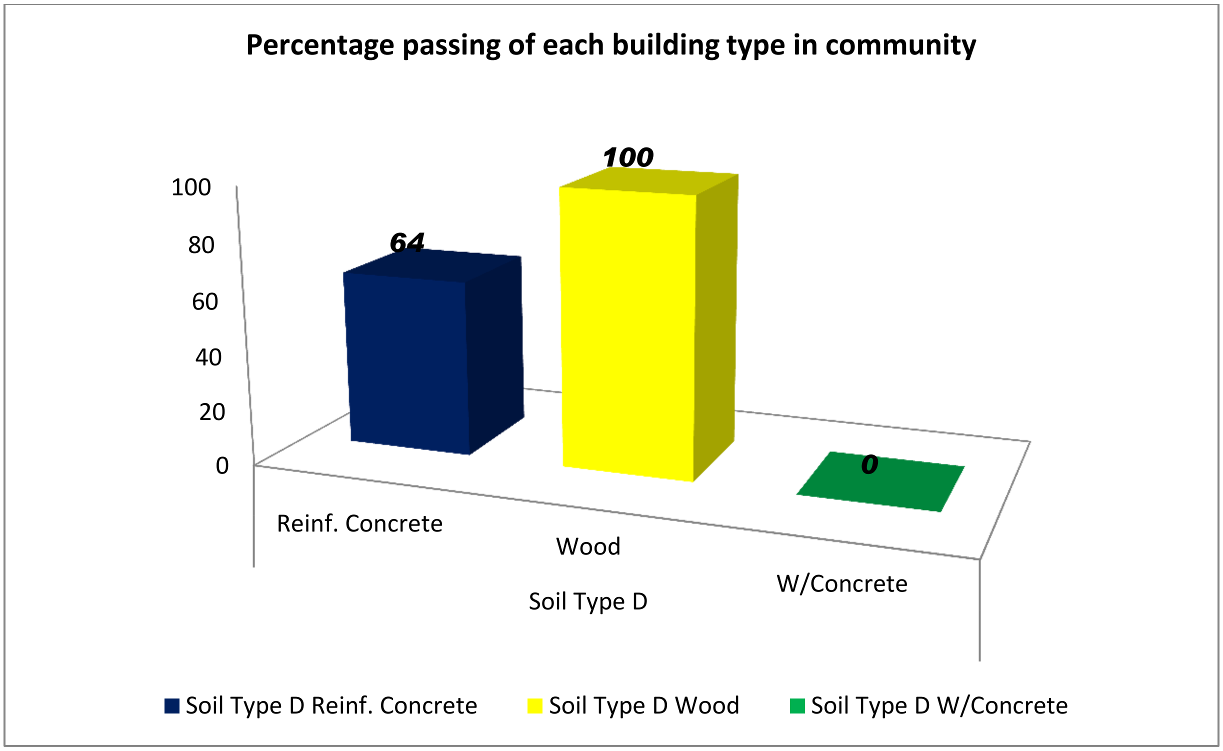


Figure 33: Bar Chart illustrating percentage passing of each building type when modifiers are applied

Wood structures are generally better seismic performers than reinforced concrete in soil type D; however, in this instance they produced lower final score. This is due to the fact that most of the wood structures in this vicinity were in deplorable conditions and as such reflected reduced performances (see Table 26).

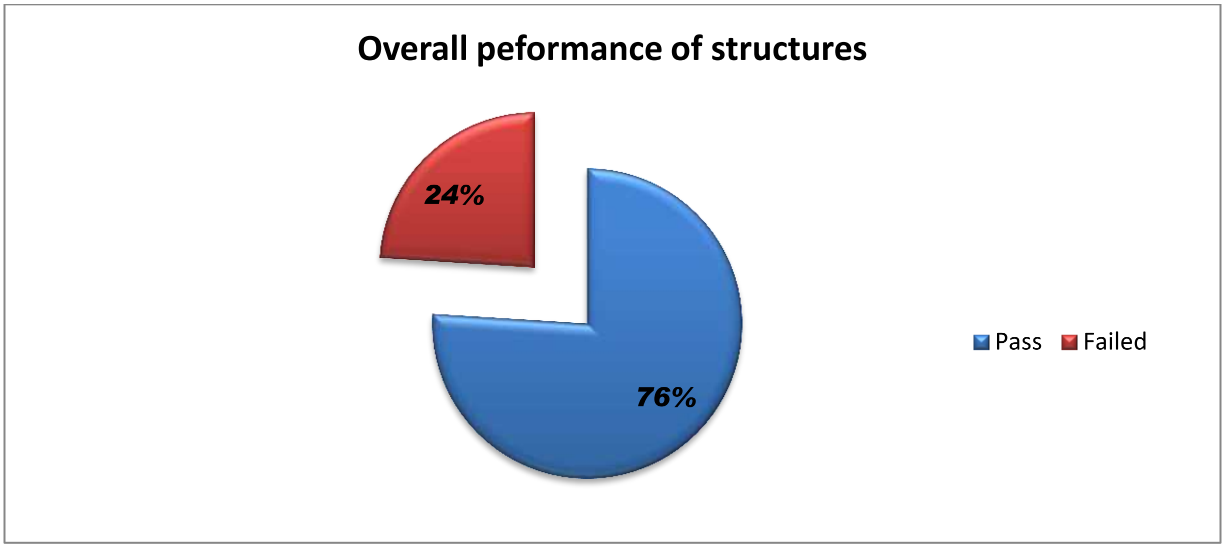


Figure 34: Pie Chart illustrating the overall performance of structures in the community

Most of the structures found in this area were reinforced concrete structures (57%) and wood structures (40%); all reflected moderate seismic performance in soil type D due to the absence of vertical irregularities. Therefore, it is seen that seventy six (76%) percent of a total of 122 buildings passed with the lowest final score produced being 3.7. The minority twenty four (24%) percent that failed was a combination of reinforced concrete and W/concrete which produced final scores ranging between 1.9 and 2.4. The major contributors to failure included the soil type (across the board), vertical irregularity (reinforced concrete) and deterioration and improper construction (W/Concrete) structures.

Table 27: Summary of performance of structures in area bordered by Iterboreale to the east and the Hospital Road to the west (A3F-A3H)

Soil Type	No. of Buildings	Building Type	Plan Irregularity(%)	Vertical Irregularity (%)	Both Vertical & Plan Irregularity (%)	No Irregularity (%)	Properly Constructed (%)	Poorly Constructed (%)	Pass %	Pass Final scores 'S'	Failed %	Failed Final scores 'S'
D	69	Reinf. Concrete	14	0	14	72	100	0	64	≥3.9	36	1.9-2.4
	49	Wood	10	0	0	90	31	69	100	≥3.7	0	-
	4	W/Concrete	0	0	0	100	0	100	0	-	100	2.0
Overall Performance	122								76	≥3.7	24	1.9 - 2.4

The area located in the vicinity of the Hospital and the Gibraltar Housing Scheme

The area is situated on both soil type C and D consisting of three building types. Reinforced concrete structures reflected moderate to very good seismic performance producing final scores of at least 3.9 and 4.3 in soil type D and C respectively. However, in circumstances where vertical irregularities were present they reflected seismic vulnerability with reduced performance score of 2.3 in soil type C, as no vertical irregularities were present in soil type D. Additionally, wood structures reflected sound seismic performance both in soil type C and D with scores of at least 4.6 and 3.9 respectively.

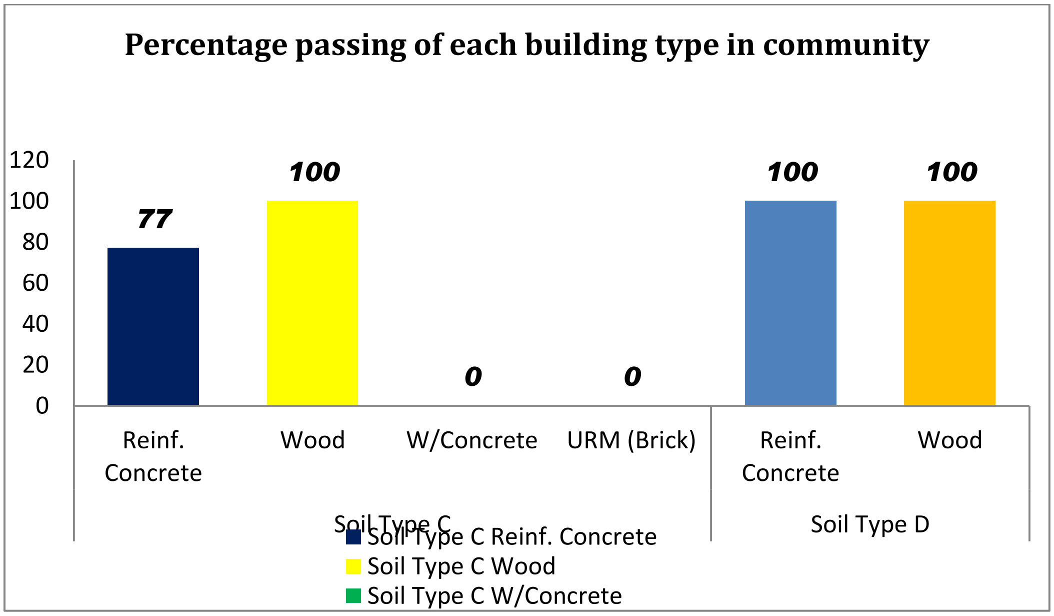


Figure 35: Bar Chart depicting percentage passing of each building between the hospital and Gibraltar Housing Scheme when modifiers are applied

Wood structures with vertical irregularities were only found in soil type C and they reflected reduced seismic performance with final scores of 2.6. Unreinforced masonry (URM) and W/Concrete structures were only present in soil type C and they both reflected poor seismic performance as result of age and deterioration (S-score = 2.1) and the presence of vertical irregularities (S-score =1), respectively (See Table 27).

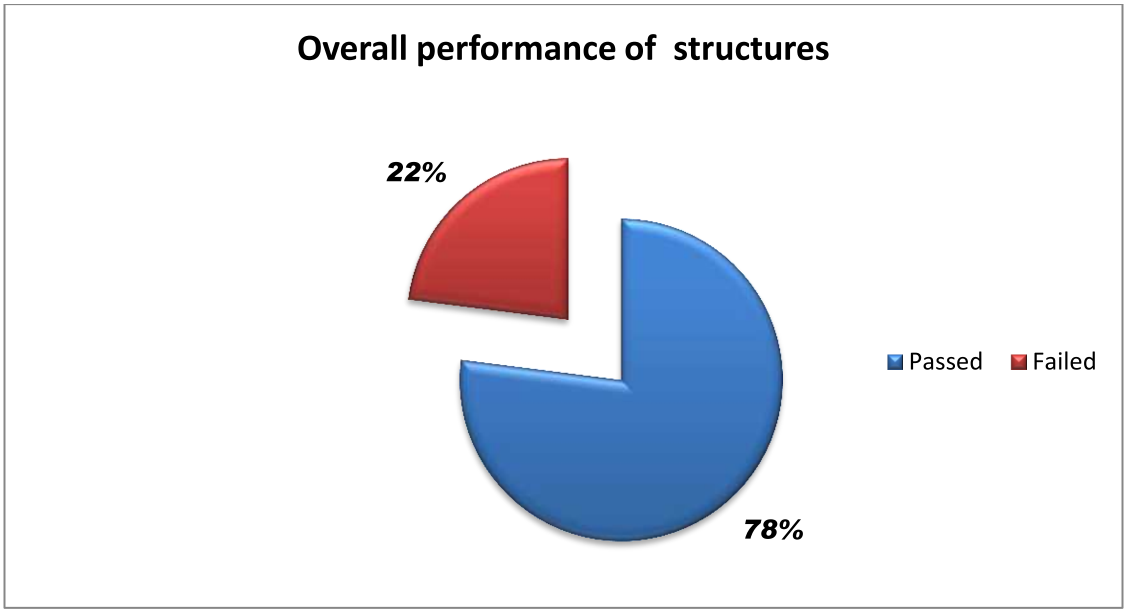


Figure 36: Pie Chart illustrating the overall performance of structures between Hospital and Gibraltar Housing Scheme.

Most of the structures found in this area (developed housing scheme) were recently constructed reinforced concrete structures which reflected on average good seismic performance in both soil types. Thus, seventy eight percent (78%) of a total of 306 buildings passed with the lowest final score produced being 3.7.The remaining twenty two percent (22%) that failed produced final scores ranging between 1.0 and 2.3 see (Figure 16). The major contributor to failure in soil type C was vertical irregularity (reinforced concrete and W/Concrete) and the pre-code factor (URM).

Table 28: Summary of performance of structures in the vicinity of the hospital and Gibraltar Housing Scheme (A3H - A3J)

Soil Type	No. of Buildings	Building Type	Plan Irregularity (%)	Vertical Irregularity (%)	Both Vertical & Plan Irregularity (%)	No Irregularity (%)	Properly Constructed (%)	Poorly Constructed (%)	Pass		Failed	
									%	Final scores 'S'	%	Final scores 'S'
C	278	Reinf. Concrete	24	0	23	53	100	0	77	≥ 4.3	23	2.3
	17	Wood	0	0	6	94	35	65	100	4.6	0	-
	3	W/Concrete	0	33	0	67	100	0	0	-	100	1.1- 2.0
	1	URM (Brick)	100	0	0	0	100	0	0	-	100	2.1
D	4	Reinf. Concrete	75	0	0	25	100	0	100	≥3.9	0	-
	3	Wood	67	0	0	75	67	33	100	≥3.7	0	-
Overall Performance	306								78	≥3.7	22	1.0 - 2.3

The area located between the Gibraltar Housing Scheme Road and Pencar River

This community was located on soil type D, with three building types namely reinforced concrete, wood and W/Concrete, with wood structures reflecting the best seismic performance (S-score 3.7 - 4.6).

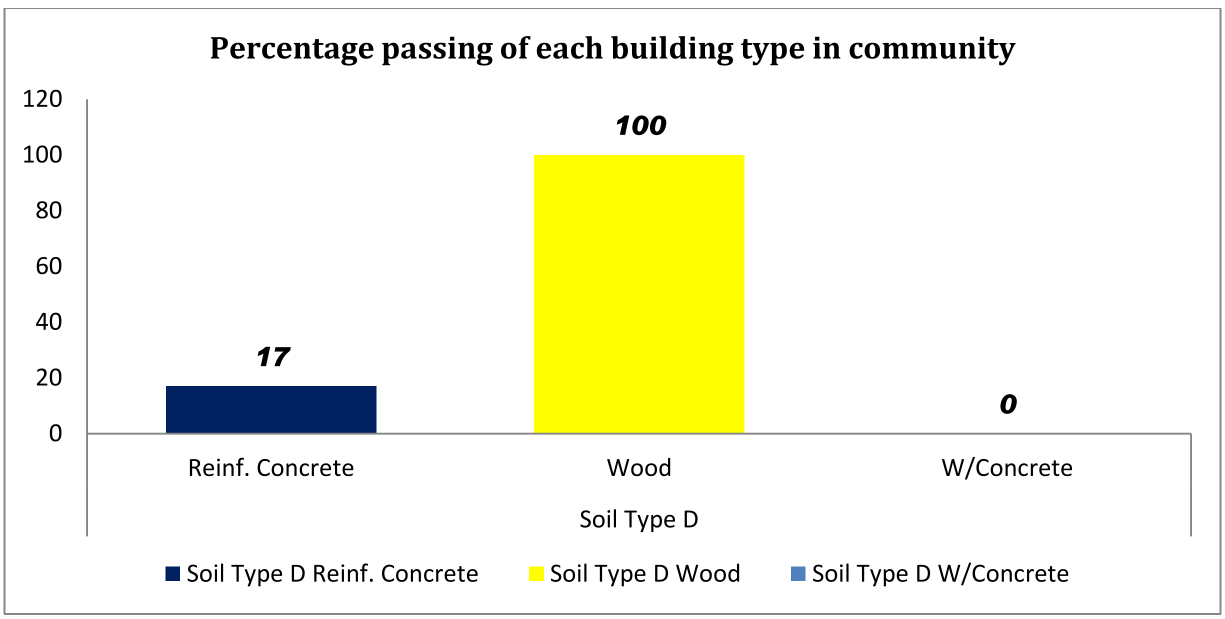


Figure 37: Bar chart depicting percentage passing of each building type between Gibraltar Housing Scheme Road and Pencar River when modifiers are applied.

Reinforced concrete constituted the larger percentage concentration of buildings but a fair amount of the existing building showed both plan and vertical irregularities which further reduce their scores on this soil type. Additionally, only 17% of these structures were recently constructed and done so properly producing final scores of 4.4. All building types located on this soil type except W/Concrete structures reflected moderate seismic performance, as these structures were poorly constructed (See Table 28).

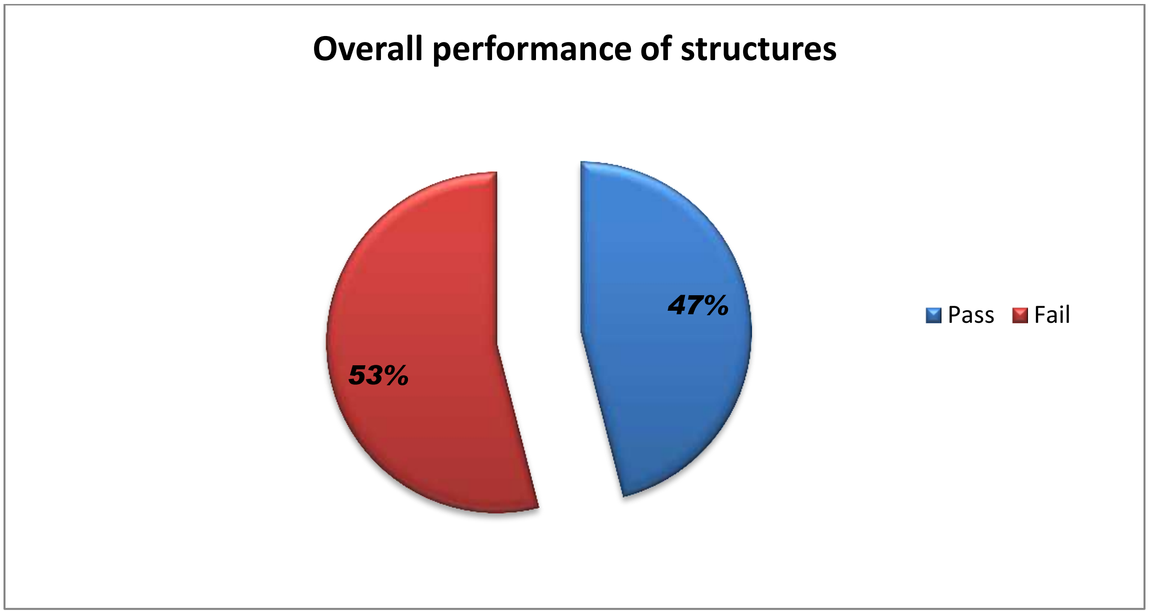


Figure 38: Pie Chart depicting the overall performance of structures between Gibraltar Housing Scheme Road and Pencar River

Reinforced concrete structures accounted for the largest percentage (62%) concentration of buildings in this area, followed by wood structures at 36%. However, 83% of a total of 84 reinforced concrete structures failed (S-score 1.9 -2.4) and thus negatively affected the overall percentage of that building type that passed. As a result, only 47% of a total of 135 buildings passed with the lowest final score produced being 3.7 .The remaining fifty three percent (53%) that failed produced final scores ranging between 1.9- 2.4 and included both reinforced concrete and W/concrete structures. Major contributors to failure included the soil type (across the board), vertical irregularities (reinforced concrete) and poorly maintained structures (W/Concrete). Complete data assessment forms can be found in Appendix C.

Table 29: Summary of performance of structures between Gibraltar Housing Scheme Road to Pencar River (A3J - Pencar River)

Soil Type	No. of Buildings	Building Type	Plan Irregularity(%)	Vertical Irregularity (%)	Both Vertical & Plan Irregularity (%)	No Irregularity (%)	Properly Constructed (%)	Poorly Constructed (%)	Pass %	Pass Final scores 'S'	Failed %	Failed Final scores 'S'
D	84	Reinf. Concrete	25	0	10	65	100	0	17	4.4	83	1.9 - 2.4
	49	Wood	8	0	0	92	31	69	100	≥3.7	0	-
	2	W/Concrete	0	0	0	100	0	100	0	-	100	2.0
Overall Performance	**135**								**47**	**≥3.7**	**53**	**1.9 - 2.4**

The area located Pencar River and Annotto River

This area is located only on soil type E and consisted of seven building types with reinforced concrete and wood structures accounting for the highest percentage concentration at 48% and 42% respectively. In this soil type only wood and containers (steel) reflected very good seismic performance with 98% (S- score 3.5- 4.0) and 100% (S-score 3.4) passes respectively. The only other building type that had passes was reinforced concrete which had 8% (S-score 1.5-2.0) of a total of 248 RM1 buildings passing due to the absence of irregularities and the application of the post code modifier. Although most of the structures were considered to be built within the post code period the post code modifier was not applied as a fair amount of these structures were observed on squatter like settlements and design and construction methods would imply the absence of adequate engineering considerations and approval. The remaining building types all reflected very poor seismic performance with all the buildings for each category failing with final scores ranging between -0.6 and 2.0. This is due to the fact that the low scores assigned to the soil type, and also some of the structures were deteriorated while others were built within the pre-code era (Table 29).

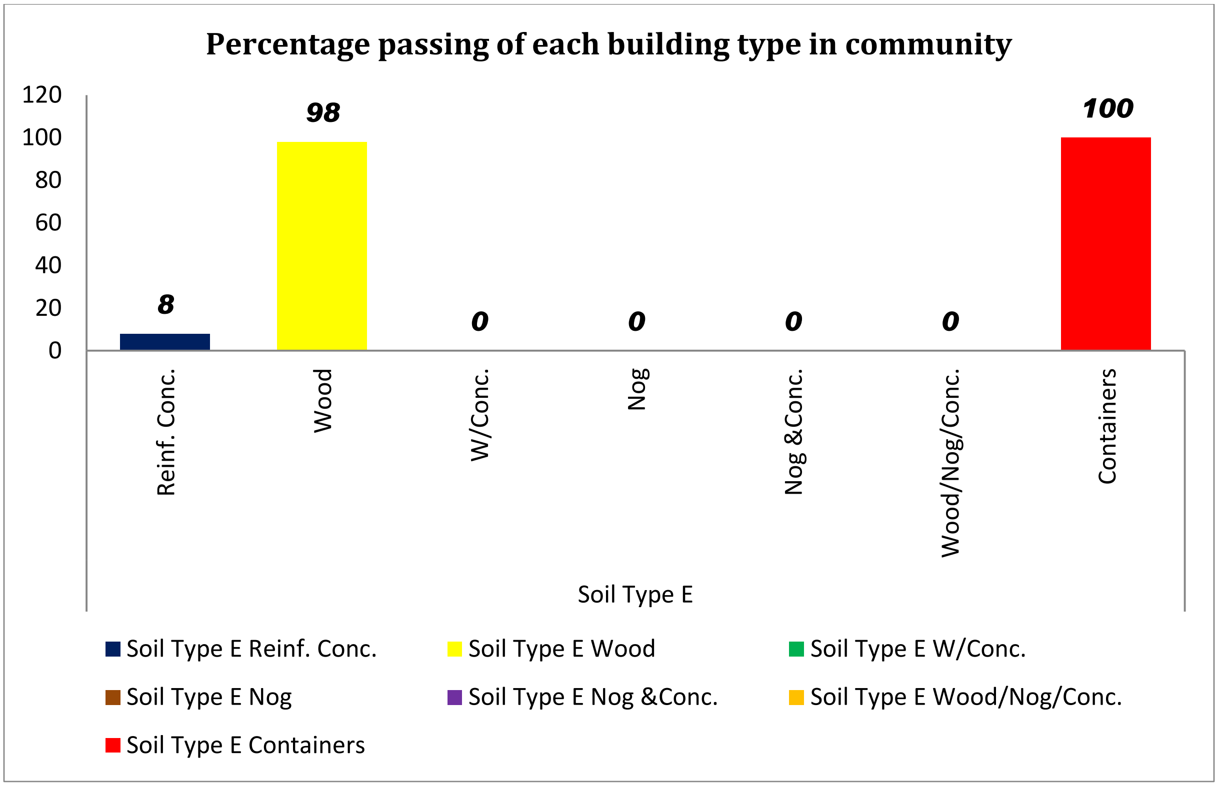


Figure 39: Bar Chart showing percentage passing of each building type between Pencar and Annotto Rivers when modifiers are applied

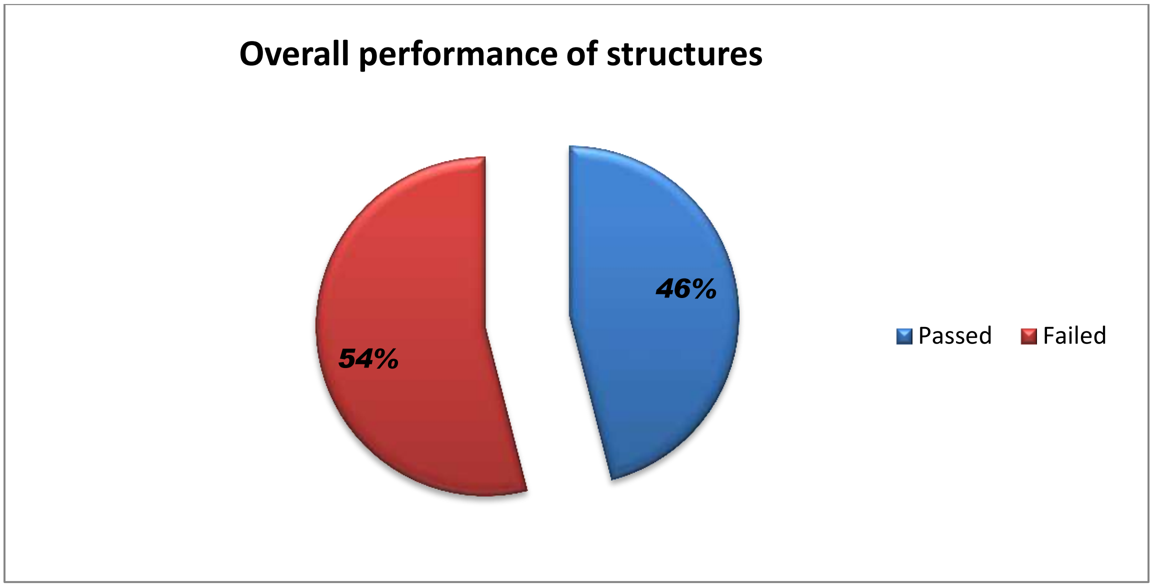


Figure 40: Pie Chart depicting the overall performance for the community between Pencar and Annotto Rivers

As RM1 buildings accounted for the highest building type concentration (46%) and only eight (8%) percent of these buildings pass the RVS. The poor seismic performance of these structures (Figure 37) negatively affected the overall percentage of buildings in this community. Therefore, only 46% of a total of 521 buildings passed with the lowest final score produced being 3.1. The remaining fifty four percent (54%) that failed produced final scores as low as - 0.6. The major contributors to failure included the soil type (across the board), vertical irregularities (RM and all building type combination with RM1) and the pre-code factor.

Table 30: Summary of performance of structures in the community located between Pencar and Annotto Rivers

Soil Type	No. of Buildings	Building Type	Plan Irregularity(%)	Vertical Irregularity (%)	Both Vertical & Plan Irregularity (%)	No Irregularity (%)	Properly Constructed (%)	Poorly Constructed (%)	Pass %	Pass Final scores 'S'	Failed %	Failed Final scores 'S'
E	248	Reinf. Concrete	24	0	5	71	100	0	8	4.0	92	1.5 - 2.0
	216	Wood	16	0	2	82	39	61	98	≥3.5	2	0
	45	W/Concrete	64	0	0	36	67	33	0	-	100	1.5-2.0
	1	Nog	0	0	0	100	100	0	0	-	100	1.8
	1	Nog &Concrete	0	0	0	100	0	100	0	-	100	1.3
	1	Wood/Nog/Conc.	0	0	100	0	100	0	0	-	100	-0.6
	9	Containers	0	0	0	100	0	100	100	3.4	0	-
Overall Performance	521								46	≥3.1	54	-0.6

The area extending from Annotto River to Fire Station End

This community is located only on soil type E, with four building types namely; reinforced concrete, wood, W/concrete and nog, with percentage concentration being 48%, 41%, 10%,1% respectively. Of the four building types only wood structures reflected sound seismic performance (100% passes) with final scores ranging between 3.5 and 3.6. Conversely, the remaining three building types all had 100% failure with final scores ranging between 1.5 and 2.0 (See Table 30).

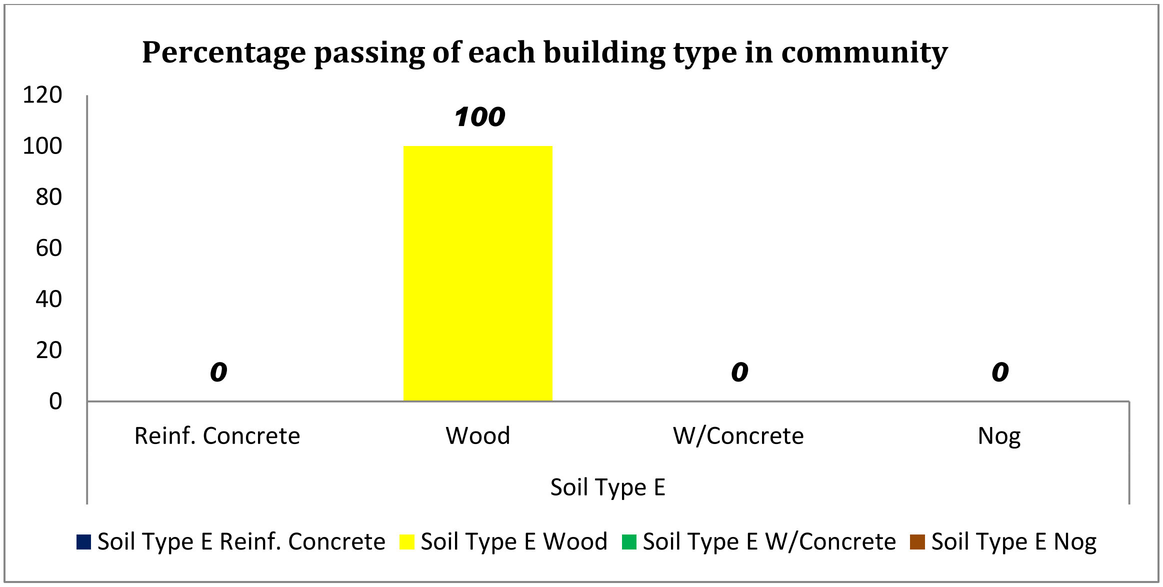


Figure 41: Bar Chart showing percentage passing of each building type from Annotto River to Fire Station End when modifiers are applied

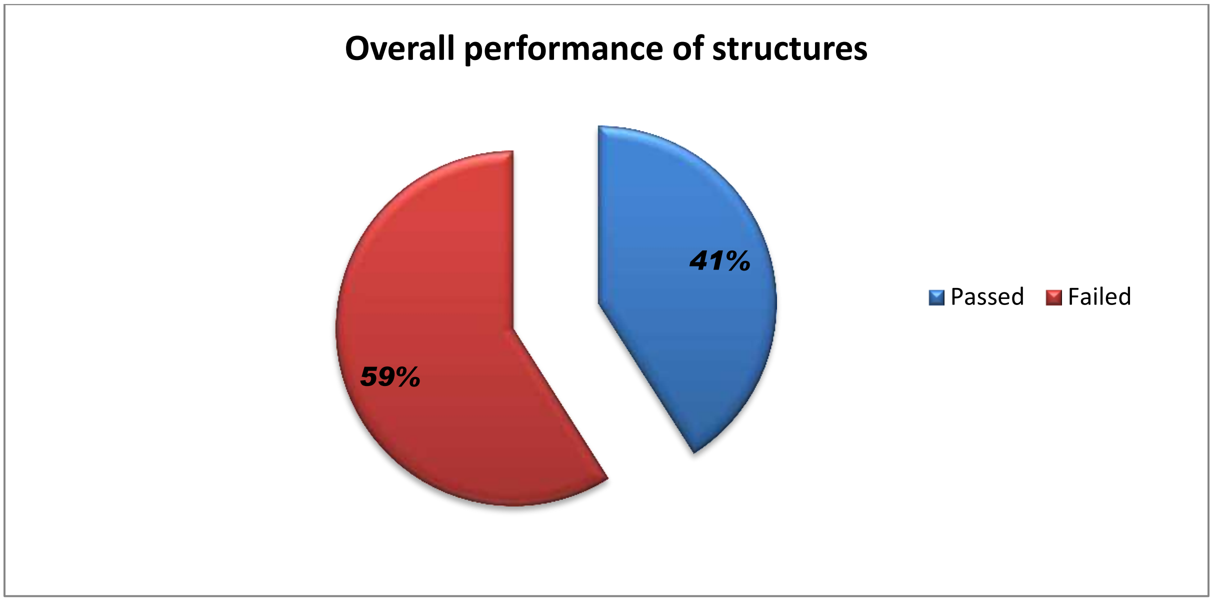


Figure 42: Pie Chart illustrating the overall performance of structures from Annotto River to Fire Station End

As previously mentioned, only wood structures in this community reflected good seismic performance with all passing (100%). As such their 41% concentration was reflected in the overall 41% of buildings that passed with the lowest final score produced being 3.5, the remaining 59% that failed produced final scores between 1.5 and 2.0. The major contributor to failure was the soil type, vertical irregularity (reinforced concrete), plan irregularity and pre-code (W/Concrete) and the pre-code factor (Nog).

Table 31: Summary of performances of structures in community extending from Annotto River to Fire Station End

Soil Type	No. of Buildings	Building Type	Plan Irregularity(%)	Vertical Irregularity (%)	Both Vertical & Plan Irregularity (%)	No Irregularity (%)	Properly Constructed (%)	Poorly Constructed (%)	Passed		Failed	
									%	Final scores 'S'	%	Final scores 'S'
E	52	Reinf. Concrete	48	0	13	39	100	0	0	-	100	1.5 - 2.0
	45	Wood	29	0	0	71	11	89	100	≥3.5	0	-
	11	W/Concrete	36	0	0	64	9	91	0	-	100	1.5-2.0
	1	Nog	0	0	0	100	0	100	0	-	100	1.8
Overall Performance	109								41	≥3.5	59	1.5 - 2.0

The community of Grays Inn

This community is located on soil type D, with five building types namely; reinforced concrete, wood, W/concrete, URM (brick) and wattle and daub, with percentage concentration being 49%, 46%, 3%, 1% and 1% respectively. Wood, URM and Wattle & Daub structures were the only building types that reflected good seismic performance giving 100% passes with final scores of at least 2.6. This was a result of 97% of the wood structures and 100 % URM and wattle &daub having no irregularities.

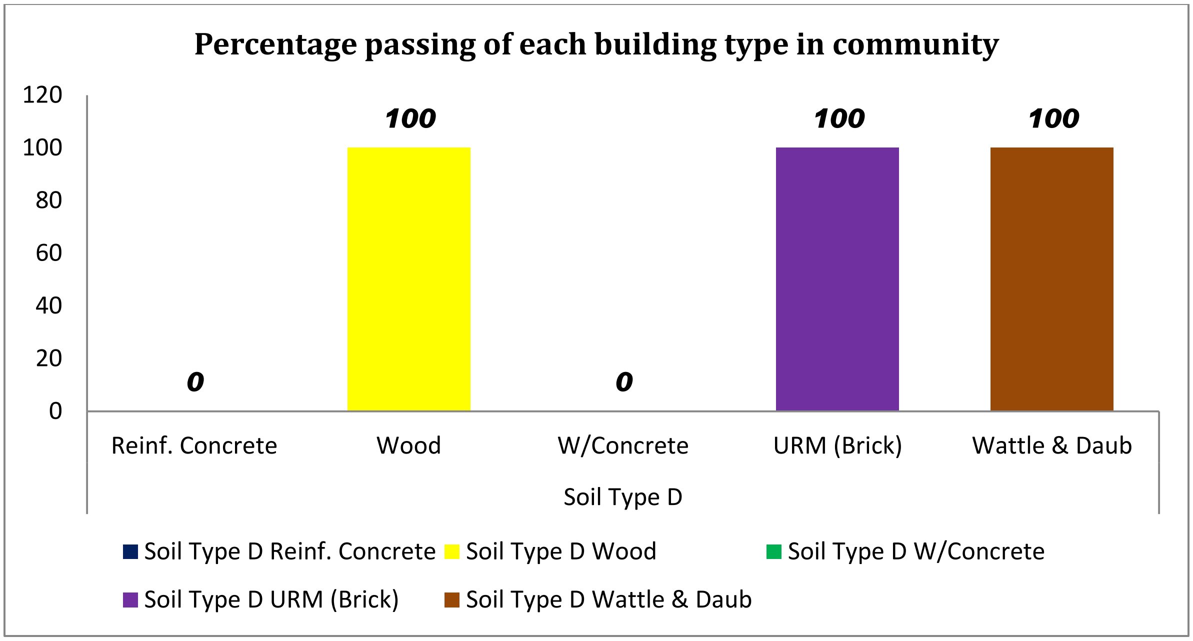


Figure 43: Bar Chart showing percentage passing of building types in the community of Grays Inn based on soil type

Conversely, the remaining two building types all had 100% failure with final scores between 1.5 and 2.4. Apart from the soil type being detrimental on the seismic performance of these structures, most of the reinforced structures were also affected by the pre-code factor while some of the W/Concrete structures were improperly constructed (See Table 31).

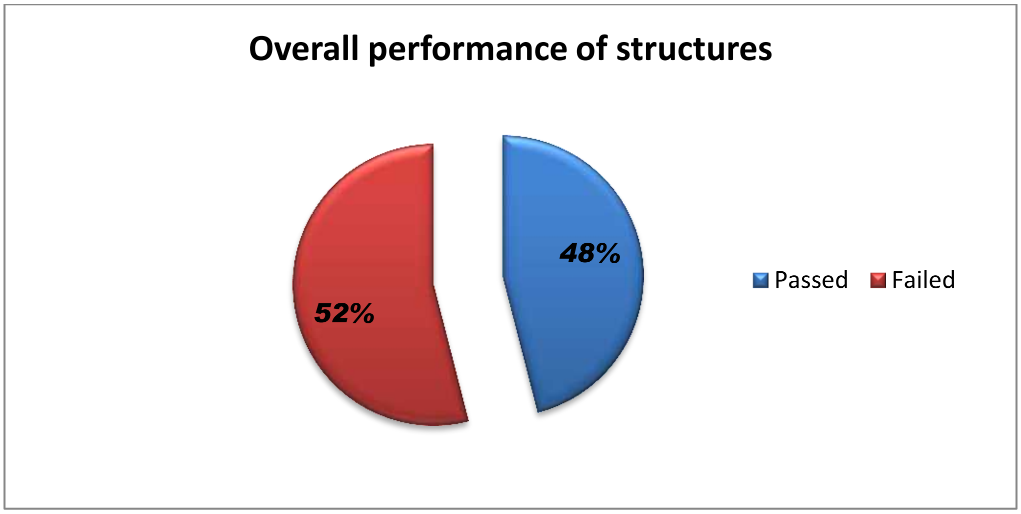


Figure 44: Pie Chart illustrating the overall performance of structures in Grays Inn

As wood, URM and wattle & daub structures were the only building type to produce sound seismic performances (100%), their respective 46%, 1% and 1% percentage concentration determined the overall 48% buildings that passed (S-score 2.6 – 4.6). The remaining 52% that failed comprised of the remaining two building types with final score between 1.5 -2.4. The major contributors to failure included the soil type, deterioration of structures and the pre-code factor.

Table 32: Summary performance of structures in the community of Grays Inn

Soil Type	No. of Buildings	Building Type	Plan Irregularity(%)	Vertical Irregularity (%)	Both Vertical & Plan Irregularity (%)	No Irregularity (%)	Properly Constructed (%)	Poorly Constructed (%)	Pass %	Pass Final scores 'S'	Failed %	Failed Final scores 'S'
D	53	Reinf. Concrete	21	0	0	79	100	0	0	-	100	1.9 - 2.4
	50	Wood	4	0	0	96	14	86	100	≥3.7	0	-
	3	W/Concrete	67	0	33	0	67	33	0	-	100	1.5-2.0
	1	URM (Brick)	0	0	0	100	100	0	100	2.6	0	-
	1	Wattle & Daub	0	0	0	100	100	0	100	2.6	0	-
Overall Performance	**108**								**48**	**≥2.6**	**52**	**1.5 - 2.4**

4.6.2 Vulnerability of Critical Facilities- RVS Performance

In the study area, a total of eight (8) critical facilities according to FEMA – 154 methodology were screened as shown in Table 32.

Table 33: Summary performance of critical facilities in Annotto Bay

Summary of Critical Facilities			
Name	**Type of Occupancy**	**Final Score**	**Pass/Fail**
Annotto Bay All Age	School	≥ 1.6	Fail
Annotto Bay High	School	≥ 2.7	Fail
Annotto Bay Hospital	Emergency	≥ 1.5	Fail
Annotto Bay Court House &Tax Office	Government	≥ 1.4	Fail
Annotto Bay Fire Station	Emergency	3.5	Pass
Annotto Bay Health Centre	Emergency	3.5	Pass
Annotto Bay Police Station	Emergency	≥ - 0.4	Fail

St. James Basic School	School	1.5	Fail

The Annotto Bay All Age School consist of three building types RM1, C1 and S1. These structures are located on soil type E and had no irregularities; however, the RM1 and S1 structures were built in the pre-code era. These produced low finals scores of 1.6 (both RM1 and S1) and 2.6 (C1). These had signs of structural defects which also prompted for more detail analysis (See Appendix B).

The Annotto Bay High School consisted of two building types RM1 and C1. These structures were located on soil type D and had plan irregularities. These structures were built within the post-bench mark era; however, signs of major structural defects in the form of major cracks were observed. These produced least finals scores of 3.9 and 2.7 respectively.

The Annotto Bay Court House & Tax Office was a URM (Nog) structure located on soil type E. This structure was constructed in the pre-code era and had no irregularities. Therefore the final score produced was 1.4.

There were four (4) critical facilities that consisted only of RM1 structures and these included the Annotto Bay Hospital, Fire Station, Library, Heatlh Centre. All of these structures except the Hospital (located on soil type D) were located on soil type E. The Hospital buildings had plan irregularities and were constructed in the pre-code era. Additionally, major structural defects were also observed and included major cracks, exposed reinforcement and one observed dislocation of a column from a building due to soil movement. Final scores that were produced by these structures ranged from 1.5 and 2.0 (See Annex B). The Fire station, Library and the Health centre were all built in the post-benchmark era and had only plan irregularities. Therefore the final score produced by all these structures was 3.5. The basic school was located on soil D and were constructed in pre-code era.

The Annotto Police Station was the facility that had more than two building types and these included RM1, URM (Brick) and W1. This facility was located on soil type E and included both plan and vertical irregularity. This facility produced the lowest final scores of all the critical facilities and included 1.5, -0.6 and -0.4 for RM1, URM (Brick) and W1 respectively.

Finally, out of the eight (8) critical facilities that were screened only two (2) (approximately 25%) of these structures passed (no detailed assessment is necessary), while the remaining eight (approximately 75%) failed (detailed assessment is necessary). The least final scores produced for passes and failures were 3.5 and -0.6 respectively; see figure 26.

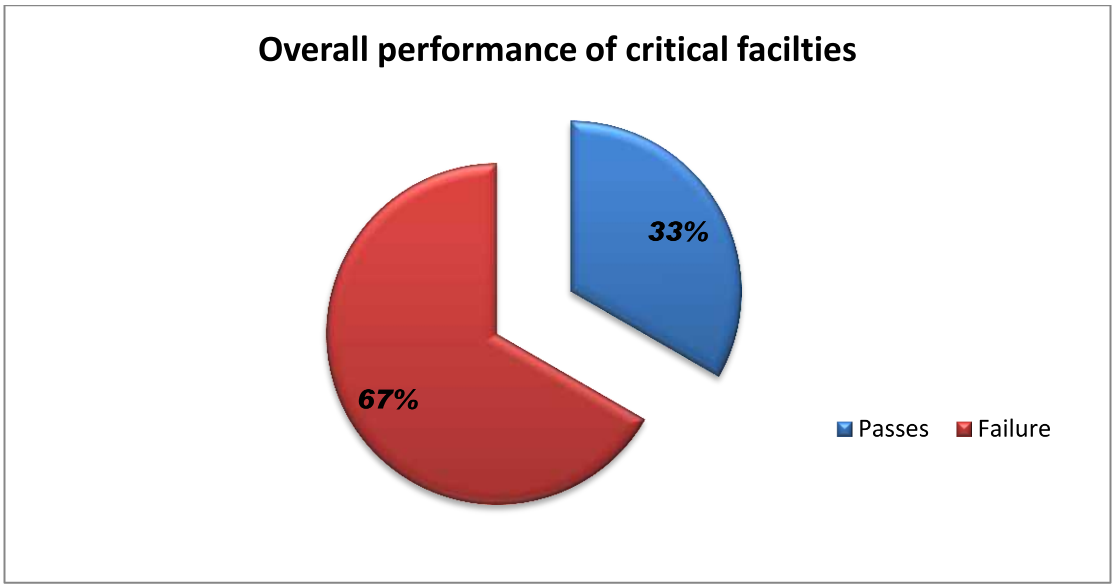


Figure 45: Pie Chart showing overall performance of critical facilities in Annotto Bay

4.6.3 Summary of RVS Results

Overall performance of the building types showed that containers and the wattle & daub (only found in Grays Inn) were the best seismic performers in the study area producing 100% passes; however, it must be noted that these building types only constituted 0.6% and 0.1% of the total number of buildings. The next best seismic performer was wood structures which gave 99% passes out of a concentration of 29.6%. Wood structures have proved to be good seismic performers in all soil types if vertical irregularities are not present, which was mostly the case in this area. Reinforced Concrete structure follows wood structures with 47% of the building passing out of a concentration of 64.8% and then URM structure with 50% passing out of a 0.1% concentration.

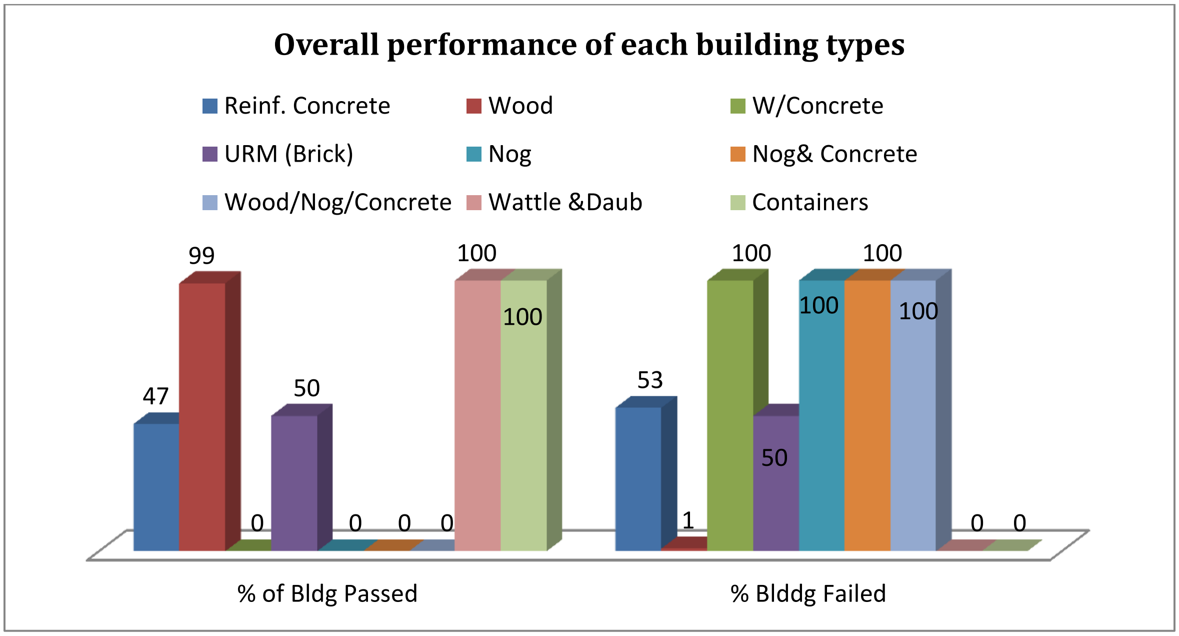


Figure 25: Column chart showing overall performance (% passes and failure) of building types in study area.

Table 34: Distribution of buildings in the community and the percentage of structures passing RVS

Community	Number of Buildings	Percent passing based on RVS
Iterboreale	197	86
Community between Iterboreale and Hospital	122	76
Vicinity of Hospital and Gibraltar Housing Scheme	306	78
Gibraltar Housing Scheme to Pencar River	135	47
Pencar River to Annotto River	521	46
Annotto River to Fire Station	109	41
Community of Grays Inn	108	48

4.7 Economic Vulnerability to Hazards

Economic losses in Annotto Bay will vary and thus depend on the nature and severity of the hazard impact. The town centre as aforementioned is very flat and so the penetration of flood water via rivers and storm surges has the potential to cause serious damage and losses. Losses will be associated with structural or non-structural damage, damage to contents, and interruption of business activities due to damage buildings, short term disruption of business activities and capital costs of repair. Economic activities in the town mainly comprise of financial institutions (National Commercial Bank and Jamaica National), wholesales, restaurants and bars, variety stores, pharmacies, a number of plazas, taxi operators as well as vendors. On the outskirts of the town centre in a south westerly direction is the St. Mary Banana Estate/JP Foods which is the largest producer of banana for export and local market in Jamaica.

Geographic Information System (GIS) based analysis shows that approximately 105 of 146 commercial buildings are exposed to hurricane induced storm surges as they are situated in the 100 year modeled surge boundary (See Figure 25). This represents of 72% of all commercial buildings in the community which has serious implications in terms of monetary losses that could be sustained by the economic sector in the community, especially since the PIOJ soci-economic profile indicated that the parish is one of the poorest in the country. Table 34 shows the modeled flood depth associated with each of the storm surge scenarios.

Table 35: Number of Commercial Buildings Exposed to Storm Surge Scenarios

Storm Surge Scenario	Flood depth (m)	Number of Commercial Buildings
25 YR RP	1m-1.6m (3ft- 5ft)	93
50 YR RP	1.2m-1.8m (4ft-6ft)	103
100 YR RP	1.5m-2.m (5ft- 7ft)	105

Similar to storm surge, the town centre is also vulnerable to riverine flood. The town is sandwiched between the floodplain of two (2) rivers and the 2001 flood event affected 109 commercial buildings for approximately 2 days. Agricultural lands in Annotto Bay are also vulnerable to crop damage, especially the banana farm of the St. Mary Banana Estate. In the case of the 2001 flood event approximately 27.6 acres of agricultural lands for banana cultivation, Banana Packaging Plant and the Green house were inundated with flood waters up to 4 feet. Moreover, another 154.7 acres of agricultural lands were also inundated. Tropical storms and hurricane winds also cause significant damage to banana plantation and small farmers often resulting is high monetary losses.

Regarding seismic vulnerability, the findings of the RVS has indicated that only 8% of reinforced concrete structures between the Pencar and Annotto Rivers passed. This area primarily constitutes the town centre which sits on alluvial deposits and this type of soil will amplify seismic waves during an earthquake. There are 45 reinforced concrete commercial buildings in this area; which may perform poorly during major seismic events. That is, these structures are deemed seismically hazardous and require detailed engineering assessment. The National Commercial Bank, one of the major financial institutions in the community failed the RVS methodology because the building had both plan and vertical irregularities and these factors (especially vertical irregularity) along with the soil factor are very detrimental to seismic performance and as such produced a final score of 1.5. The credit Union on the other hand had neither plan nor vertical irregularities and was also produced in the post-benchmark era. This building produced a final score of 4.0 which was the highest of all the critical facilities.

Based on the H/V assessment there is a clear indication that sections of the town of Annotto Bay should show high ground amplification during a major earthquake, the section in the downtown area close to the police station courthouse are areas where highest amplification is expected and also show characteristics of deep soil thickness. These are potentially unstable areas and as such any building design in these areas must take into consideration the potential for resonance.

This vulnerability to multi- hazards can significantly affect the economic stability of the community and in particular the agriculture sector which is a major employer and contributor to the local economy and the country's GDP.

4.8 Summary of Vulnerability for Annotto Bay

The table below summarizes the results of the vulnerability analysis for the 3 hazard types and elements at risk. The values show the figures for each hazard type and the specific element at risk in the community.

Elements at Risk	Hazard		
	Storm Surge	Riverine Flooding	Earthquake
Buildings	621	729	854
Population	2010	2740	2219[5]
Roadway (km)	1.9	3.5	
Crops (hec)	-	74	-

[5] This figure represents the population between Pencar and Annotto Rivers only because this area is susceptible to liquefaction. The population representing the buildings that failed the RVS is not included in the summary table because the data is aggregated and so it is difficult to estimate the corresponding population that reside in seismically hazardous buildings.

5.0 RISK ASSESSMENT

5.1 Introduction

Risk is defined as the combination of the probability of an event and it negative consequences (UNISDR, 2009). Risk can also be defined as the probability of harmful consequences or expected losses (deaths, injuries, property, livelihoods, economic activity disrupted or environment damaged) resulting from interactions between (natural or human-induced) hazards and vulnerable conditions in a given area. This risk assessment will focus on tangible losses that is, things that have a monetary (replacement) value, for example buildings and infrastructure.

5.2 Defining Depth Damage Function (Vulnerability Curves)

Once the elements at risk are identified it is possible to assess how they would be impacted by hazards using vulnerability curves (Jonkman et al. 2008 and Broekx et al 2011). Vulnerability curves are constructed on the basis of the relation between hazard intensities and damage data. They provide a relation in the form of a curve, with an increase in damage for a higher level of hazard intensity. Different types of elements at risk will show different levels of damage given the same hazard intensity.

Damage due to storm surges depend on several factors such as direct exposure to wave action, water depth, debris and so on. This risk assessment focuses on damage related to water depth only and this is done through the use of vulnerability curves that estimate the direct impacts of flooding from storm surge. While it is preferable to develop a unique set of vulnerability curves for each community or country, the data requirements to do so limit the feasibility of developing such geographically specific stage damage curves (Davis et al. 2003). However, the relationship between flood depth and damage to a given asset is relatively consistent justifying the use of generic curves. While the ODPEM is not aware of any curves specifically developed for use in Jamaica, the U.S. Army Corps of Engineers have developed generic curves that can be used throughout the United States to assess the impacts to residential structures and content (USACE 2010) and the California Department for Water Resources Flood Depth Analysis for all building types (CDWR, 2012). The depth damage curves developed by CDWR correspond to 1 and 2 storey government buildings and the associated percentage content damage.

For a building to be included in the analysis the flood water had to have a height above, or equal to, the structure's first floor. For each flood depth, the percent damage to each individual structure is estimated by applying an existing structural depth-damage curve for government buildings developed by the California Department for Water Resources Flood Depth Analysis (CDWR 2012). Figure 46 shows the stage damage that was used to determine the vulnerability of the critical facilities and used in the calculation of risk. Damage starts subsurface at -0.5m which is associated with possible damage to the foundation of these facilities.

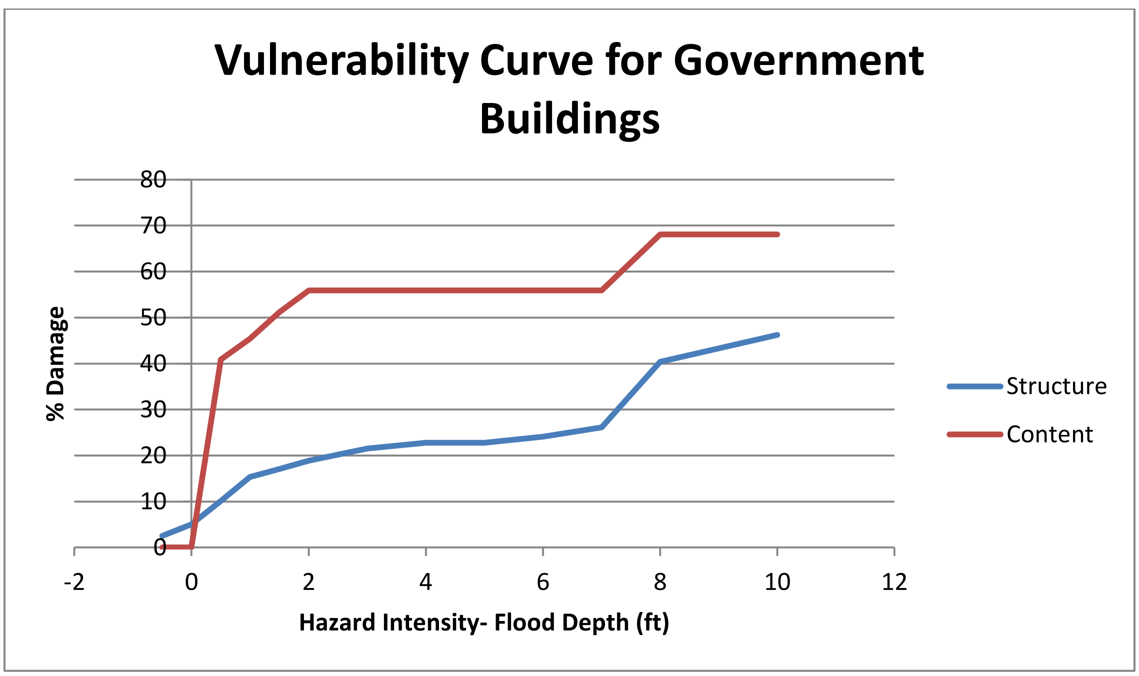


Figure 46: Vulnerability Curve for Critical Facilities

5.3 Probabilistic Risk Assessment

The methodology used for the calculation of risk is based a quantitative approach which aims at quantifying risk according to the following equation:

$$R_S = \text{Hazard} * \text{Vulnerability} * \text{Amount of elements at risk}$$

$$R_S = P_T * P_L * V * A$$

Equation 1

P_T- is the temporal (e.g. annual) probability of occurrence of a specific hazard scenario within a given return period;

P_L- is the locational or spatial probability of occurrence of a specific hazard scenario with a given return period in an area impact the elements at risk;

V- is the physical vulnerability, specified as the degree of damage to a specific element at risk given local intensity due to the occurrence of hazard scenario.

A -is the quantification of the specific type of element at risk evaluated. It is important to indicate that the amount can be quantified in different ways and that the way the amount is quantified is also the same way the risk is quantified. For example, the amount can be given in numbers such as the number of buildings (the risk is then the number of buildings that might suffer damage), number of people (e.g. casualties/injuries/affected) and also in economic terms i.e. monetary losses.

Loss estimates provided in this multi-hazard assessment are based on best available data, and the methodologies applied result in an approximation of risk. Moreover, due to inadequacy and unavailability of data, expected losses are calculated only for the critical facilities in the study area. These estimates should be used to understand relative risk from hazards and potential losses.

The expected losses will not be expressed as individual buildings but on aggregations of all the critical buildings that are at risk. Presenting risk information at building level has implications for real estate values and insurance. Moreover, the hazard information is not so detailed to indicate risk for every individual building and detailed characteristics of each building is required.

Uncertainties are inherent in any loss estimation methodology, arising in part from incomplete scientific knowledge concerning natural hazards and their effects on the built environment. Uncertainties also result from approximations and simplifications that are necessary for a comprehensive analysis (e.g., incomplete inventories, demographics or economic parameters).

5.4 Storm Surge Risk Calculation

For the study area, risk is calculated or expressed as both the number of buildings potentially at risk to storm surge as well as losses associated with the scenarios for storm surge. The economic loss results are presented using annualized losses which is the expected loss per year when averaged over a long period of time (e.g. 100 years). In other words, the estimated annualized loss addresses the key issue of risk which is represented as the amount of money that has to be paid in the long term to offset the losses associated with storm surge. The annualized losses are calculated using the following three (3) steps:

1. Compute/estimate losses for a number of scenario events with different return periods [e.g., 10-year, 25-year, 50-year, 100-year];

2. The expected losses per scenario are plotted against the temporal probability of occurrence in a graph. Through the points a curve is fitted, called risk curve;

3. Calculate the area under the fitted curve to obtain annualized losses.

Table 21 in Chapter 4 indicated the total number of buildings that are potentially at risk to coastal flooding associated with the different return periods for storm surge. These buildings are exposed the flooding because of location in the modeled hazard footprint (See Figure). These buildings are exposed and therefore at risk of suffering damage during storm surges. From Table 35 it can be seen that the number of buildings exposed to storm surge hazard increases with each return period or the annual probability of occurrence of each scenario. The annual probability is calculated as the reciprocal of the return period.

Table 36: Number of Buildings at Risk to Storm Surge Scenario

Return Period	Annual Probability	Number of Buildings at Risk
10	0.1	490
25	0.04	548
50	0.02	585
100	0.01	621

The number of buildings at risk is represented on risk curve by plotting the number of buildings at risk of sustaining damage against annual probability as indicated below in Figure 45.

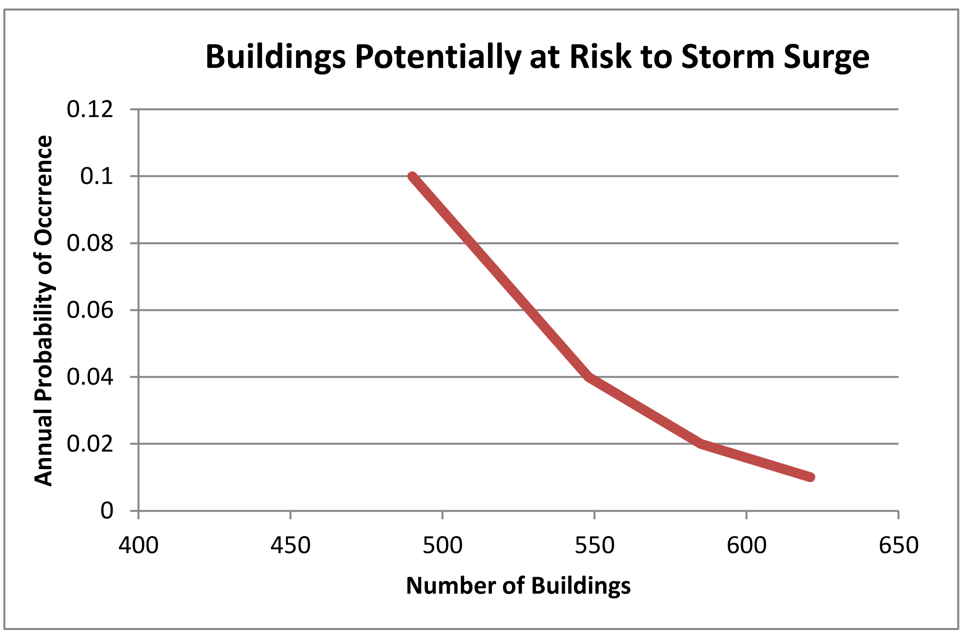


Figure 47: Buildings Potentially at Risk to Storm Surge Impact

Vulnerability for each structure is determined by the depth damage curve (See Figure 46) associated with the given flood depth. The total value of the structure is the combination of both the replacement and content value of the buildings which were ascertained in consultation with the critical facilities. The content value for Dorcas Basic and ABC Smart Kids represents a fraction of the content value for Annotto Bay All Age which is determined by the ratio or fraction of the building footprint (square footage) of the basic schools to that of the Annotto Bay All Age. The content value for the health centre represents the value of computer equipment, furniture and fittings etc for the Annotto Bay Hospital.

Potential flood losses from storm surge were estimated for the seven (7) critical facilities and the assessment of costs focused on tangible losses.

Table 37: Total Value of Critical Facilities in Annotto Bay

Element at Risk (Critical Facilities)	~Building Footprint (M^2)	~ Replacement Value ($JMD)	Content Value ($JMD)	Total Value of Structure at Risk ($JMD)
Dorcas Basic School	315.22	18,913,200	475,063	**19,388,263**
Annotto Bay All Age School	1459.77	102,184,012	2,200,000	**102,404,012**
Fire Station	267.560	20,334,560	3,781,239	**24,115,799**
Police Station	263.66	22,993,500	296,519	**23,290,019**
Inland Revenue/Court	266.99	25,630,944	-	**25,630,944**
ABC Smart Kids Learning Centre Learning	306.58	22,993,500	462,042	**23,455,542**
Annotto Bay Health Centre	980.98	68,668,670	9,714,534	**78,383,204**

Loss estimates are based on the probabilistic scenarios for storm surge for the five (5) return periods (See Table 37) and is obtained by multiplying the vulnerability (percent damage as determined by vulnerability curve) of critical facility (V) for the specific scenario by the total value (A) of the structure (V*A). Total losses are then calculated by summing all the losses of storm surge scenario for all the elements at risk (critical facilities) exposed to the scenario. As discussed above, the vulnerability for each building is derived from stage damage curve which shows the relationship between depth of water and percentage damage.

Table 38: Expected Losses Based on Storm Surge Scenario

Scenario	Return Period	Annual Probability (P_T)	Expected Losses (V*A) $JMD millions
Storm surge_5y[6]	5	0.2	0
Storm surge_10y	10	0.1	28.3
Storm surge_25y	25	0.04	61.0
Storm surge_50y	50	0.02	94.6
Storm surge_100y	100	0.01	105.0

The risk curve shows the total consequences or losses associated with the storm surge scenarios for all the elements at risk in this case, critical facilities exposed to the scenario. It is important to convert the risk curve into Averaged Annualized Loss (AAL), which as indicated earlier is the expected loss per year when averaged over a long period of time. The total annual risk is the total area under the risk curve of which the X-axis displays losses (monetary values) and the Y-axis displays the probability of occurrence. The points on the curve represent the losses associated with the return periods for which an analysis was done. The area under the curve was calculated using triangles and rectangles method.

Figure displays the relationship between the expected losses in monetary value and storm surge annual probability of occurrence with the area under the curve representing the **expected average annualized losses of JMD$ 13,007,517**. The average annualized losses

[6] The assumption is that the 5 year storm surge event will not result in any physical damage as the predicted storm surge for this return period is 1.35m which is below the elevation of the ground (~ 1.5m) relative to mean sea level.

correspond generally to the economic value that has to be paid annually in the long term to offset losses associated with future storm surge events.

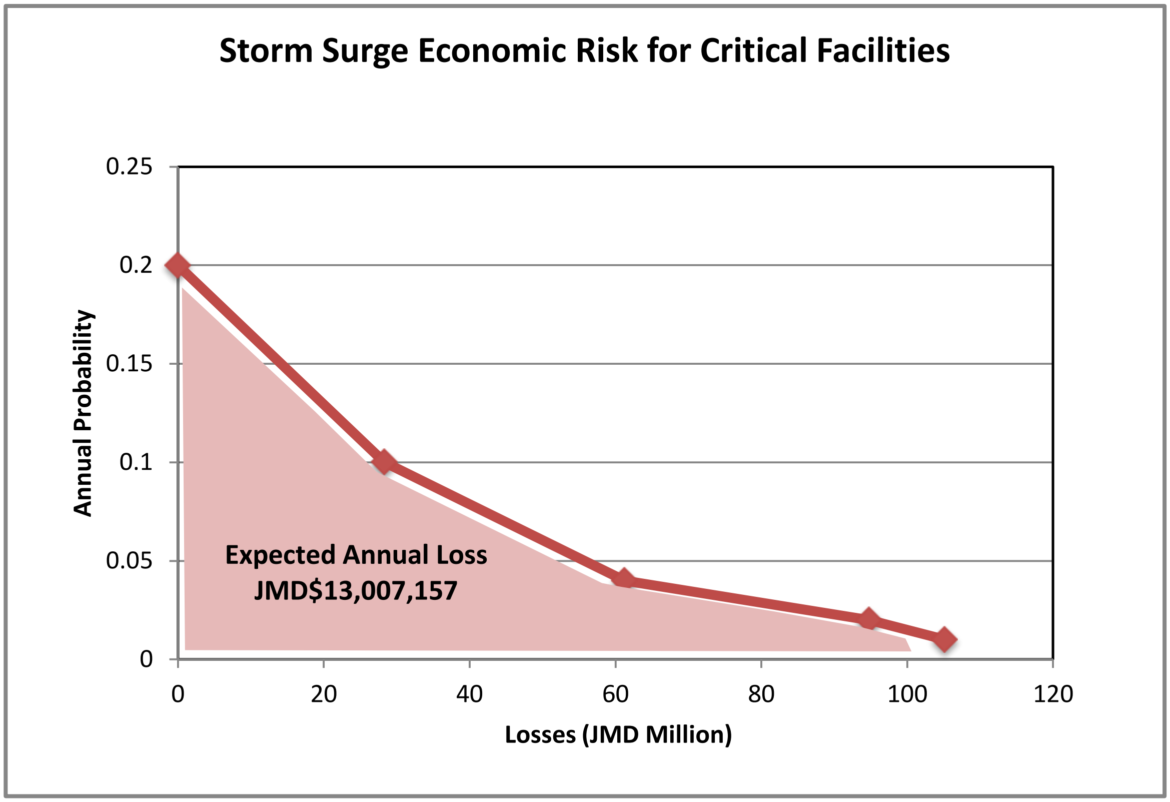


Figure 48: Economic Risk Associated with Storm Surge Hazard

It should be noted the total expected annual loss does not represent total losses for all the elements at risk but just for critical facilities. Therefore, the total expected loss for Annotto Bay is expected to greater given the exposure and vulnerability of the community to multiple hazards.

5.5 Limitations and Way Forward

1. Due to factors such as unavailability of detailed data for example, the replacement and content values of structures the risk assessment only included expected monetary losses for the critical facilities. Ideally, a comprehensive risk assessment should incorporate all of the 621 buildings that are located in the 100 year storm surge return period. Therefore, results for the expected losses do not represent the total risk for the community associated with storm surge.

2. The depth damage curve used to determine vulnerability factor of the buildings is based on the USA system which is not necessarily a reflection of the conditions in Jamaica. As such, the percentage damage associated with flood depth may in fact be more or less and would have an influence on the results. Notwithstanding, research has indicated that

Way Forward

3. Develop damage or vulnerability curves for various buildings types showing the relationship between hazard intensity and percentage damage. This information would improve accuracy of the curve (s) as they would be based on local conditions and actual flood events which can be used to simulate damage for potential events.

4. For storm surge, at least two (2) curves should be developed. One (1) for properties on the sea front as they will be more quickly destroyed due to direct exposure to wave action action and flood waters. The send would be for properties further inland will be exposed to what is referred to as “resting” water damage (Genevose, et al, 2011Develop damage or vulnerability curves for various buildings types showing the relationship between hazard intensity and percentage damage. This information would improve the accuracy as curve (s) is based on

5.0 RECOMMENDATIONS

This chapter identifies the mitigation strategies that are necessary to prevent and/or minimize the impact of natural hazards to which the community is vulnerable. It is important that the strategies/measures address vulnerability comprehensively to not only reduce the future impact of hazards but enhance the adaptive capacity of the community to climate change and sea level rise.

It is envisaged this multi-hazard risk assessment report will provide the following benefits and allow users to:

i. Identify vulnerable areas in Annotto Bay that may require special considerations;
ii. Develop simulation exercises based on hazard analysis to assess the level of readiness and preparedness of the local authority and the community ;
iii. Estimate potential losses before or after a disaster based on detailed inventory of assets in the community;
iv. Decide on how to allocate resources for most effective and efficient response and recovery;
v. Prioritize mitigation options that need to be implemented to reduce future impact and potential losses.

6.1 Mitigation Options

Mitigation options are categorized as structural and non-structural, both of which are deemed vital to comprehensively reduce the vulnerability of the community

6.1.1 Non-structural

- **Relocation –** Over 50 buildings were affected by storm surges during the passage of Hurricane Sandy in 2012 however; serious damage was confined to structures on the waterfront. Of the total, 7 houses were totally destroyed and another 13 sustained major damage. Based on the results of the storm surge hazard assessment, relocation should be considered as an option for those structures located 18 m from the high water mark and those that are in breach of the 30m coastal setback from the shoreline as stipulated by the National Environment and Planning Agency (NEPA).

- **Development Planning**- Special guidelines should be formulated and enforced for for development in flood zones.
- **Evacuation Plan –** detailed evacuation plan to be developed for the community.
- **Training –** continued training, public education and simulation exercises to test the response mechanism of the emergency services – fire, police and health facilities as well as update of community disaster risk management plan as well as the response mechanism.

6.1.2 Structural

The structural mitigation option has to be hazard specific and for the three (3) hazards the following are recommended:

Earthquake

- Detailed engineering assessment of the critical facilities that failed RVS methodology. This will inform the type of retrofitting that is required for each facility.

Riverine Flood

- The National Works Agency has produced a White Paper on "Comprehensive Drainage and Flood Control Scheme" for the community of Annotto Bay. This paper identified a number of structural mitigation measures:
  i. **construction of a network of dikes** along the Pencar and Annotto Rivers
  ii. **increase the capacity of the Motherford Drain** to address vulnerability of the community to riverine flood.
  iii. **Detention ponds (2)** to address flash flooding experienced in Crooked River that section of the community.

Storm surge

- Groin for drains and gully to keep the discharge to the sea clear
- Shoreline protection in the form of buried revetments for the vulnerable areas

Figure 49 below shows the conceptual design for the Motherford Drain to keep storm water discharge to the sea clear and minimize the amount of sand deposition in mouth of drain during high tide as this cause a "back flow" effect resulting in flooding.

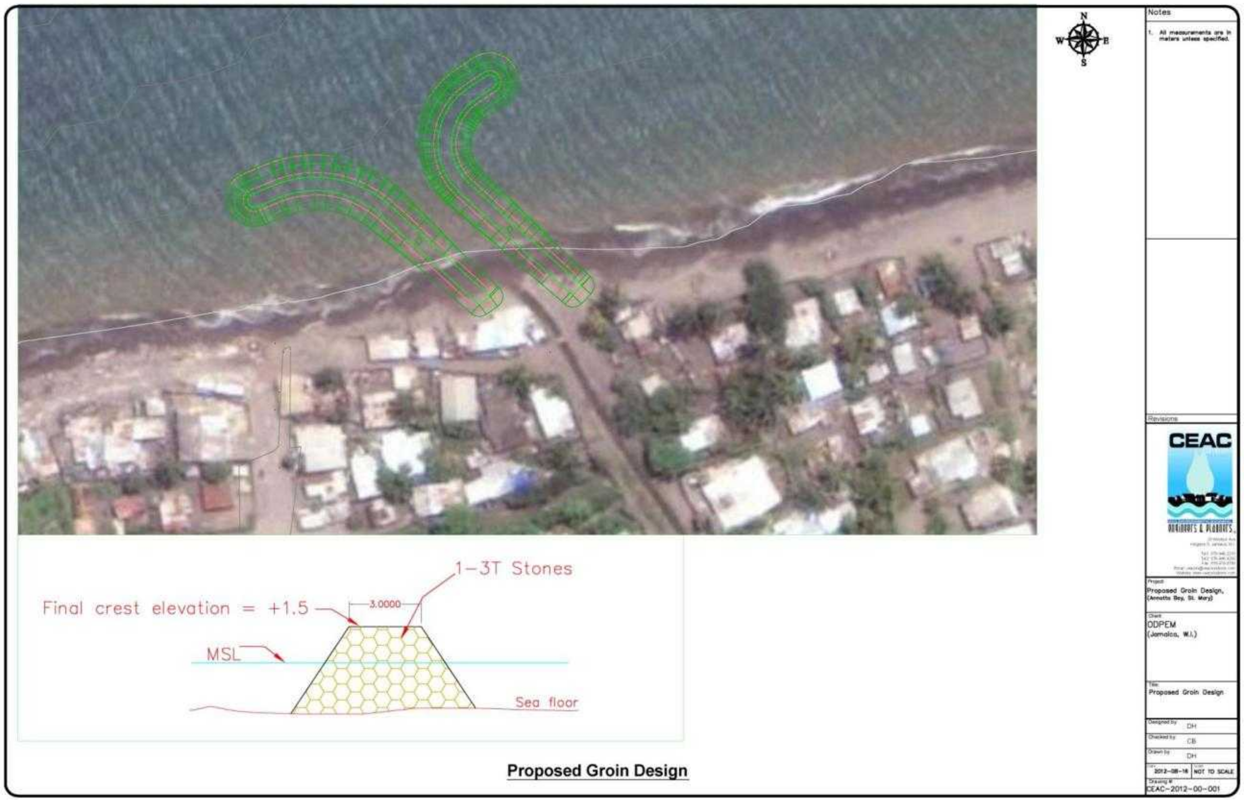


Figure 49: Schematic of proposed Groin Design

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APPENDIX

APPENDIX A – Hazard History for Annotto Bay

YEAR	DATES	EVENTS	IMPACTS	CASUALITIES
1901	December 14	Heavy Rainfall	Roads impassable; railway track flooded and wash out	None
1904	June 13	Heavy Rainfall	Banana trees blown down; Wag Water river flooded its banks; Junction Road impassible; roadways at Old Ned washed away	None
1915	August 16	Hurricane (unnamed)	▪ Over 100 families became homeless as many houses were demolished ▪ loss of lives due to drowning; ▪ below the Episcopal Mission Station every house was either completely destroyed or shifted off its foundation; ▪ 5 of the 6 “lighters” secured in the Annotto/Pencar River had been battered to pieces; ▪ The United Fruit Go.'s Wharf was dismantled and the warehouse with its stock destroyed; ▪ At the railway station many freight cars with their wheels embedded in sand ▪ Banana fields suffered severely;	4 dead
1931	May 18	Continuous Rainfall	Rivers in spate; town under water	None
1933	August 19	Heavy and Continuous Rainfall	▪ Almost every yard was covered with water ▪ Wag Water river overflowed its bank and road was impassable at Scotts bridge ▪ Cemetery was over-flooded; ▪ Railway tracks were damaged; bananas destroyed; ▪ At Jack Rock where the Banana Co. of Jamaica had a considerable amount of coal stored for running their pumping station, the house and coal were washed away.	2 dead
1940	November 21	Heavy Rainfall	▪ Encroachment of the sea; ▪ Mother Ford water contributing to a washing away of buildings; groceries, bakery and some other commercial buidlings; ▪ Houses destroyed	1 dead

YEAR	DATES	EVENTS	IMPACTS	CASUALITIES
1941	May 23	Heavy and continuous Rainfall	▪ Rivers in spate; ▪ Roads flooded and bridge blocked which made traffic impassable ▪ Houses inundated	None
1943	April 5	Heavy Rainfall	▪ Annotto/Pencar River and Miss Ford River overflowed their banks; ▪ Inundated portions of the streets and surrounding lands; ▪ People evacuated their homes; ▪ Telegraphic communication between Annotto Bay and Castleton cut off	None
1944	August 20	Hurricane (unnamed)	▪ Annotto Bay hospital completely flattened; ▪ Over 40 patients injured	None
1948	May 25	Heavy Rainfall	▪ 10 inches of rainfall which caused the water rose to a height of 6ft.; ▪ The Crooked, Pencar Miss Fords and Annotto Rivers all overflowed their banks inundating roads and houses. ▪ The Wag Water has flooded the road from the Gas Station at Agualtavale crossing to 'Scotts Bridge at Bottom Bay, emptying itself into the Annotto River bringing down heaps of cane trash from the fields, and blocking the Bridge. ▪ Residents on the Fort George Road were also flooded out, while those on Crab Hall suffered similarly. ▪ The Crooked River washed the Railway Yard, inundating the cricket field, the railway lines were also underwater. Nurses at the hospital had to take refuge in the Outpatients' Department, as their quarters were heavily	None

YEAR	DATES	EVENTS	IMPACTS	CASUALITIES
			drenched. ▪ Several cows were washed into the sea by the Wag Water. One cow drowned and was washed up at Bottom Bay. ▪ Water rising over 3 feet in all sections of Gray's Inn Factory including the storehouse, destroyed several thousand bags of sugar, and a number of bales of sugar bags and miscellaneous stores.	
1953	January 14	Heavy Rainfall	▪ Iterboreale – Enfield road inundated; ▪ Fort George road flooded; ▪ Annotto/Pencar River swollen and flooded many land	None
1954	October 12	Heavy Rainfall	▪ Annotto/ Pencar River in spate which caused homes in Bottom Bay and Port Arthur inundated; ▪ Miss Ford river inundated; roads blocked due to flooding or landslides	None
1963	Dec	Heavy Rainfall	▪ Crooked River flooded the Annotto Bay Railway Station yard. ▪ The main from Golden Grove to Iterboreale flooded.	
1966	October 29	Heavy Rainfall	Annotto Bay Public Hospital flooded	None
1966	November 4	Winds and Intermittent Heavy Rains	▪ Damage to Annotto Bay General Hospitals	None
1969	June 9	Heavy Rainfall	▪ Rivers in spate; ▪ Agualta Vale road flooded; ▪ main road from Scottsbridge at Bottom Bay to Breakneck Corner inundated	None
1980		Hurricane Allen	▪ Storm surge height of 15 feet ▪ Maximum surge distance inland 150nyrds ▪ 155 buildings damaged or destroyed	None
1988	September 12 – 13	Hurricane Gilbert	▪ Rivers flooded; roads inundated; houses flooded;	None

YEAR	DATES	EVENTS	IMPACTS	CASUALITIES
			▪ Excessively high flows of the Wag Water River caused the dislodgment of one of the four piles supporting the metal bridge near Annotto Bay.	
1999	March	Cold front	▪ Flooding occurred in this area affecting several persons. The Fort George Bridge collapsed cutting off certain transportation routes.	None
2001	October 29 – November 5	Tropical Storm Michelle	▪ Fort George Bridge collapsed, the Annotto Bay Primary lost its roof; roads were blocked; ▪ 3 rivers that traverse the town to overflow their banks resulting in damage to 307 homes.	None
2002	September 26	Tropical Storm Lili	▪ Town centre flooded with water depth 1m as the 4 drainage features in the area exceeded capacity and overflowed their banks.	None
2004	September 22	Hurricane Ivan	▪ Rivers in spate which caused flood levels of up to 1m; ▪ Houses flooded; ▪ Sections of roadways inundated	None
2009	February 9	Heavy Rainfall	▪ Annotto Bay to Fort George road is inundated	None

APPENDIX B- NEPA's Land Use Classification

Revised Land Use Categories	Land use code
Residential	
Residential Single Family	RES_SF
Residential Multifamily (Apartment/Town House)	RES_MF
Residential Informal (squatting)	RES_INF
Resort	
Hotel Motel (Inn) Guest House (Villas)	RET_H RET_M RET_GH
Commercial (Retail e.g. supermarkets, shops, markets, shops/ Restaurants, Clubs, Bars, Gas station, Barber shops/ hairdressers, Undertakers, Markets)	COM
Office (Travel agency, Financial institutions, Lawyers office, Accountants, Banks	OFF
INDUSTRIAL * Light Industry - (furniture, garment, timber yard ,bakery, garage, repair shop, upholstery shop, shoe making shop) * Heavy Industry - Oil refinery, sugar factory or other uses generating much noise, smoke, fumes, dust or traffic)	 IND_L IND_H
Public Assembly (Public worship and religious instructions e.g. churches, mosque, temple, convent etc.)	PA
Public Buildings (courts, police station, Fire Station and libraries)	PB
Educational (schools, colleges, universities	EDU

Revised Land Use Categories	Land use code
,specialized schools e.g. school of music and art)	
Institutional (art gallery, museum, health center, hospital, nursing home, day care center)	INST
Recreational (theatre, cinemas, halls, community centers, social clubs, gymnasium, racetrack)	REC
Monument/Historic Building	MM/HB
Open Space (parks, playing field, tennis & badminton court, golf courses	
* Private	OS_Pvt
* Public	OS_Pub
Cemeteries/Crematoria	CEM
Military	M
Warehouse (Attached to factory or commercial enterprise, pound for cars	W_HSE
Vacant Lot	V_Lot
Vacant Building	V_Bldg
Derelict Building	DB
Parking	
* Private	PK_Pvt
* Public/Municipal	PK_Pub/M
Transportation Centre	TC
UTILITIES * Waterworks (dams,	UTILITY

Revised Land Use Categories	Land use code
reservoir, tanks & filter plant)	U_WW
* Sub-stations (Electricity)	U_SSt
* Sewage plant	U_SP
* Pump Station/Lift Station	U_PSt
* Cell Site (Stand alone, on building, cell tower)	U_CS
* Power Plant	U_PP
* Nuclear Station	U_NS
* Wind Farm	U_WF
* Hydroelectric Plant	U_HP
Quarry	Q
Landfill/dump	LF/D
Land Cover	
Agriculture (Cropland)	AGRI
Forest	F
Grassland	GL
Shrub/Woodland	S/WL
Ruinate (Temporary Disturbed Forest)	R
Wetlands (mangrove, swamps)	W_Land
Civil Aviation (Air Port, Air Field, Aerodrome)	CA
Ports (Sea Port)	P
Embassy/High Commission/consulate	EMB
Under Construction	UC
Mix Use (three or more use)	MU
Residential/Commercial	RES/COM

Revised Land Use Categories	Land use code
Residential/Office	RES/OFF
Residential/Light Industry	RES/IND_L
Residential/Public Assembly	RES/PA
Residential/Educational	RES/EDU
Residential/Institutional	RES/INST
Residential/Agricultural	RES/AGRI
Office/Commercial	OFF/COM
Educational/Institutional Educational/Institutional	EDU/INST
Public Assembly/ Educational	PA/EDU
Commercial/Light Industry	COM/IND_L
Office/Light Industry	OFF/IND_L
Office/Educational	OFF/EDU
Public Assembly/Institutional	PA/INST
Residential Informal/ Commercial	RES_INF/ COM

APPENDIX C:

Final Assessment Report

Storm Surge and Shoreline Erosion Hazard Mapping

For

Annotto Bay, St. Mary

Prepared for:

Prepared by:

August 2012

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	Submission to Client	Revision A	Revision B
Prepared by:	DH/CJ	DH	
Reviewed by:	CB	CB	
Approved by:	CB	CB	
Date:	30/07/2012	17/8/2012	
Comments:	Submitted to ODPEM	Mitigation measure to gully added and 25 year RP	

Introduction

Background

ODPEM Office of disaster Preparedness and emergency management. as a part of their mandate to assist communities with disaster readiness has contracted the services of CEAC Solutions to prepare storm surge hazard report for Annotto Bay. Annotto bay is on the north eastern coast of Jamaica in the parish of St. Mary. The town in situated in a low lying area (varying in elevations from 1 to 3 metres typically above Mean Sea Level) which is prone to flooding both from overland stormwater flows and storm surge from the sea.

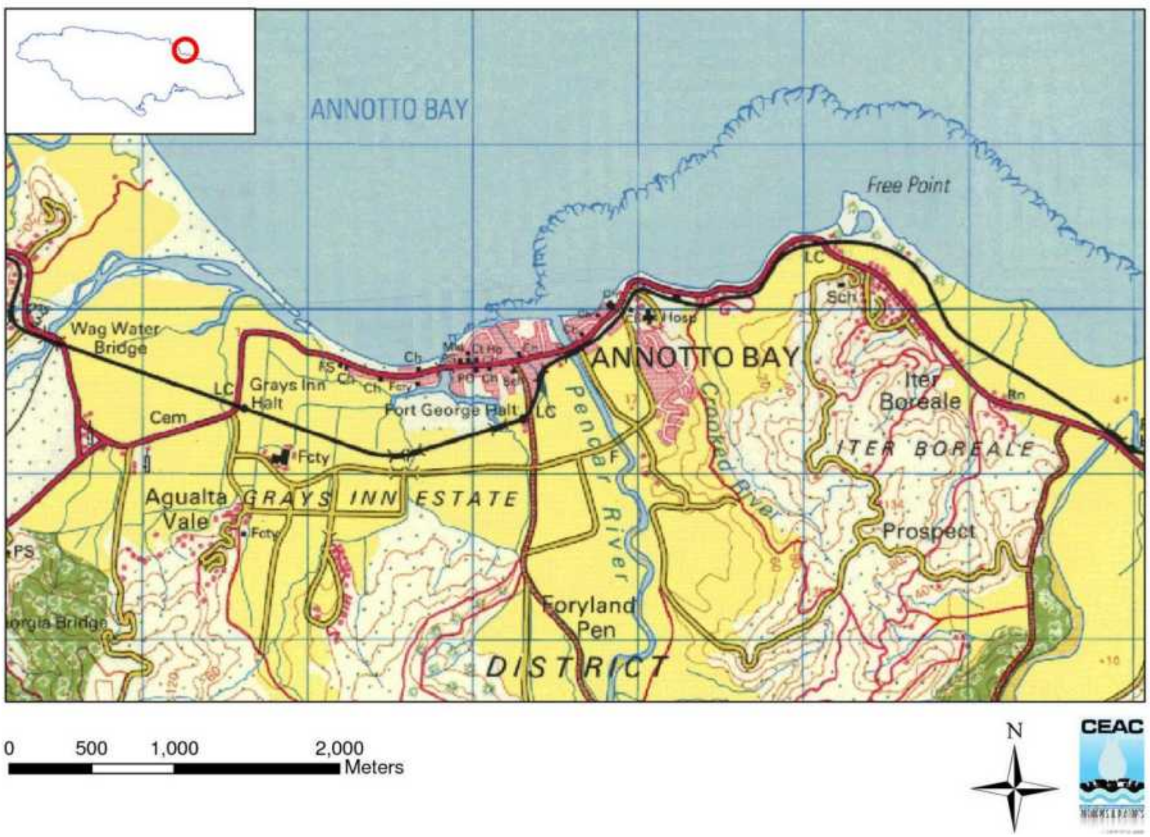


Figure 0.1 Location plan of Annotto Bay

Storm Surge

Storm surge is an increase in water levels generated by a storm, which is above the normally expected astronomical tides. This rise in water level can cause extreme flooding in coastal areas particularly when storm surge coincides with normal high tide, resulting in storm tides which can reach levels of up to 20 feet or above mean sea level.

Storm surge is has two components, they are the wind driven surge and the pressure driven surge. The former is produced by water being pushed toward the shore by the force of the winds moving cyclonically around the storm whilst the latter is produced by the low pressure associated with intense storms. The pressure driven component of the surge is however typically around 5% of the total surge.

Figure 0.2 Wind and Pressure Components of Hurricane Storm Surge, Source: National Hurricane Centre, NOAA

Storm climate affecting Jamaica

IPCC (2007) projections for climate change in Tropical storms are for more intense storms.

"There is evidence from modelling studies that future tropical cyclones could become more severe, with greater wind speeds and more intense precipitation. Studies suggest that such changes may already be underway; there are indications that the average number of Category 4 and 5 hurricanes per year has increased over the past 30 years.

The predictions of the IPCC are consistent with the number of category 4 and 5 storms that have tracked within 400 kilometres Jamaica in the past 130 years. This determination was made by querying the National Oceanographic and Atmospheric Administration Hurricane Centre database on hurricane tracks for storms that pasted within 400 kilometres of Jamaica shorelines. Nodes off the north, south, east and west coast was used. See Figure 0.3 which clearly shows that the number of category 4 and 5 storms has increased from 10 to 15 storms per twenty year interval up to 1950 to 30 to 35 storms per twenty year after 1950. This doubling of storm occurrences coupled with increased sea level rise can result in shoreline retreat as beach profiles adjust to the more intense wave climate.

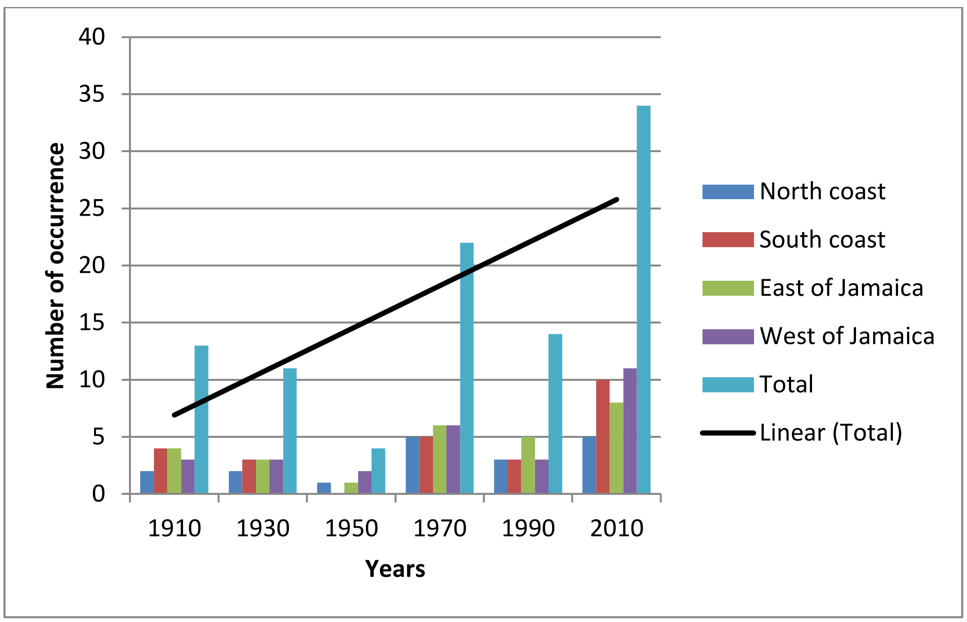


Figure 0.3 Occurrences of Category 4 and 5 hurricanes that have passed within 300 kilometres of Jamaica's shoreline since 1890 to 2010, in twenty years intervals

The observations of increase hurricane frequency for extreme hurricanes (Category 4 and 5) in the recent past would suggest that the predictions of storm surge based upon observations are conservative. In other words the observations of the residents of Annotto Bay would be observations in a period of extreme hurricane climate and not during a period of relative calm.

Objectives and Scope of work

Objective

It was determined through discussions with ODPEM that the objective of this study is to determine for Annotto bay the 10, 50 and 100 year storm surge levels.

Scope of Work

The approved scope of work is as follows:

1. Field works/Data Collection in conjunction with ODPEM:
    a. Collection of anecdotal information from residents on recent storms
    b. Survey levelling to coordinate locations of observations with mean sea level
    c. Sand sampling of shoreline for three representative locations
2. Storm surge analysis
    a. Documentation of historical storm surge events and anecdotal information
    b. Calibration and verification of storm surge analysis
    c. Analysis of 10, 50 and 100 year return period storm surge levels without run-up and with run-up, using the following methodology:
        I. Definition of extreme deepwater wave climate offshore Annotto Bay.
            a. The National Hurricane Center (NOAA) database of hurricane track data in the Caribbean Sea will be used to carry out a hindcast, followed by a statistical analysis to determine the hurricane: waves, wind and set-up conditions. The data base goes back to 1886.
            b. Extraction of Storms and Storm Parameters passing within a search radius of 300km radius of the site.
            c. Application of the JONSWAP Wind-Wave Model in order to estimate the wave conditions, in conjunction with the Young Parametric Hurricane wave model for the rotating hurricane wind field.
            d. Extremal Statistics analysis of maximum wave heights using Weibull's distribution.
            e. Directional analysis of the waves approaching the site to determine the most severe direction historically.
        II. Nearshore transformation of extreme wave heights
            a. Nearshore storm surge analysis, using a profile from deep to shallow waters, for the major storm surge components, including: Wave breaking and shoaling; Wind set-up; Refraction; Tides; Global Sea Level Rise (over a 50 year project life); Inverse Barometric Pressure Rise
            b. Estimation of run-up storm surge component for three select profiles for Annotto Bay
3. Report preparation and submission to ODPEM of estimated storm surge levels for: 10, 50 and 100 year return period.

Methodology

The approach to completing this project was as follows:

1. Anecdotal Data collection on storm surge levels in the Annotto Bay communities by conducting vox pop surveys
2. Model to determine wave parameters and storm surge levels by using numerical models and historical hurricane data collected by the National Oceanic and Atmospheric Administration (NOAA)
3. Analysis of anecdotal storm surge data and comparison to model results
4. Conduct nearshore transformation modelling of waves using numerical model
5. Plot vulnerability charts and prepare report

Data Collection and Analysis

Hurricanes

Data on hurricanes from 1851 to 2010 was retrieved from the NOAA database for use in a hindcast hurricane model. The data includes for each hurricane; track points and dates for positions, central surface pressure, wind speed and intensities. This data is integral for analyzing the historical hurricanes and performing extremal statistics to determine the return periods.

Topographic Survey

In order to establish the existing shoreline and the back of beach elevations, a rapid topographic survey was conducted within the existing beach area by CEAC engineering technicians. The surveys extended from the shoreline through the residential developments to the north coast highway. The topographic data points were gathered relative to msl by surveying the shoreline and making correction for tidal fluctuations using the British admiralty tidal predictions for Port Antonio. The resulting topographic points revealed the general area flat to gently sloping up from the shoreline towards the highway. The highest elevations measured in the project area were up to 3.5m above mean sea level in the eastern section of the project area.

The topographic surveys were supplemented with elevation data from the NLA 12,500 dataset from which a digital terrain model was created to represent the actual ground surface. This DEM is however preliminary and is useful for strategic planning purposes.

Bathymetric Data

Bathymetric data forms the basis for wave transformation modelling and storm surge modelling to a lesser degree. Understanding the movement of currents along the seafloor aids in the prediction of wave intensity and direction on the shoreline. The bathymetric data used for this project was obtained from contour data provided by the client and supplemented by points and contours from the British admiralty chart 255- Eastern Approaches to Jamaica. The contours and points were digitized and used to create a digital terrain of the seafloor from shoreline to deepwater. The bathymetry of the area north of the project site has a fairly constant drop off to the edge of the continental shelf which ends 0.75km offshore at approximately 20m depth.

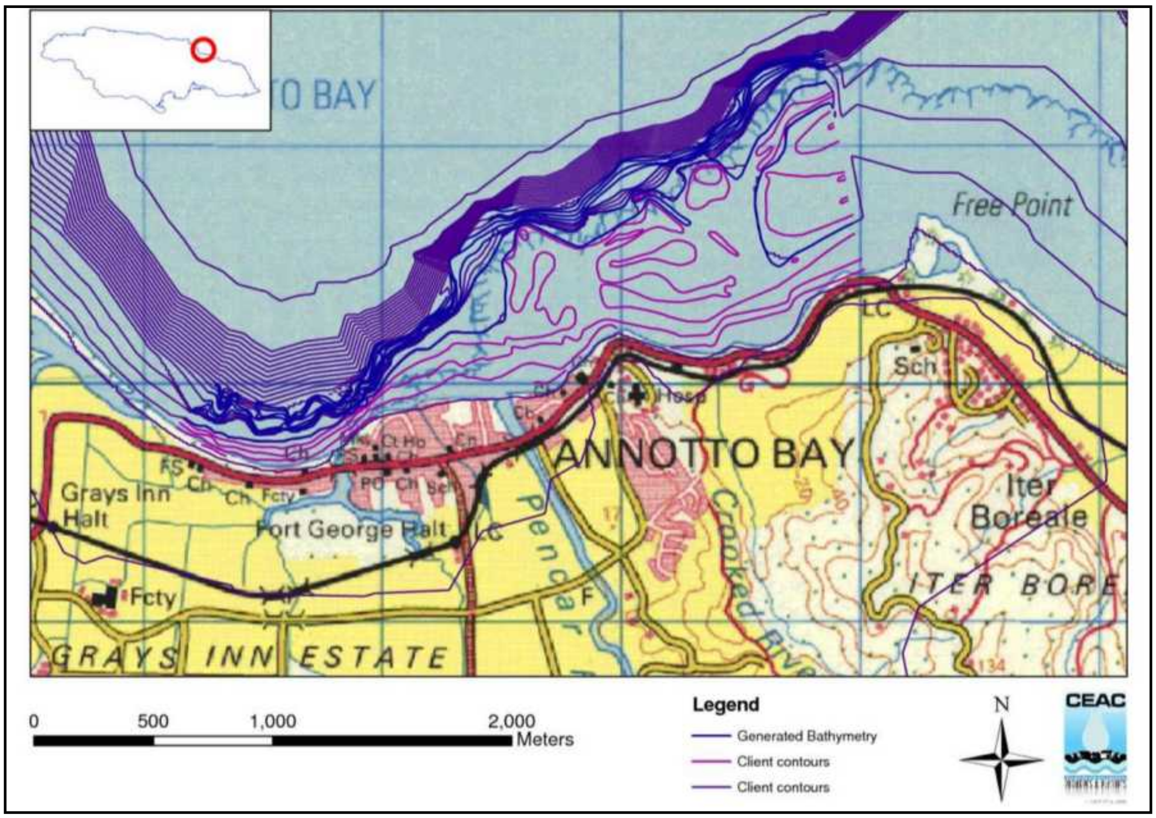


Figure 0.4 1in50000 map of Jamaica highlighting the major bathymetric features north of the project site

Sediments and Grain size Analysis

Sieve analysis

It was necessary to determine the representative grain size on the shoreline in order to understand how the beach will react to the hurricane waves. The Four (4) surface samples were collected from the project beach face and analyzed to determine the representative grain size and distribution. See Figure 2.2 below for the sediment sample location points.

The grain size analysis was done using the unified classification which is widely used for classification of granular material. The samples had median grain sizes varying from 0.413mm to 1.465mm in diameter. The classification of these samples therefore varied from medium to very coarse sand.

See Figure 2.3 and Table 2.1 below for summarized results of the analysis.

Table 0.1 Grain size analysis on samples along an the Annotto Bay shoreline

Sample ID	**Location #1**	**Location #2**	**Location #3**	**Location #4**
Mean (mm)	1.024	1.465	0.413	0.644
Description	very coarse sand	very coarse sand	medium sand	coarse sand

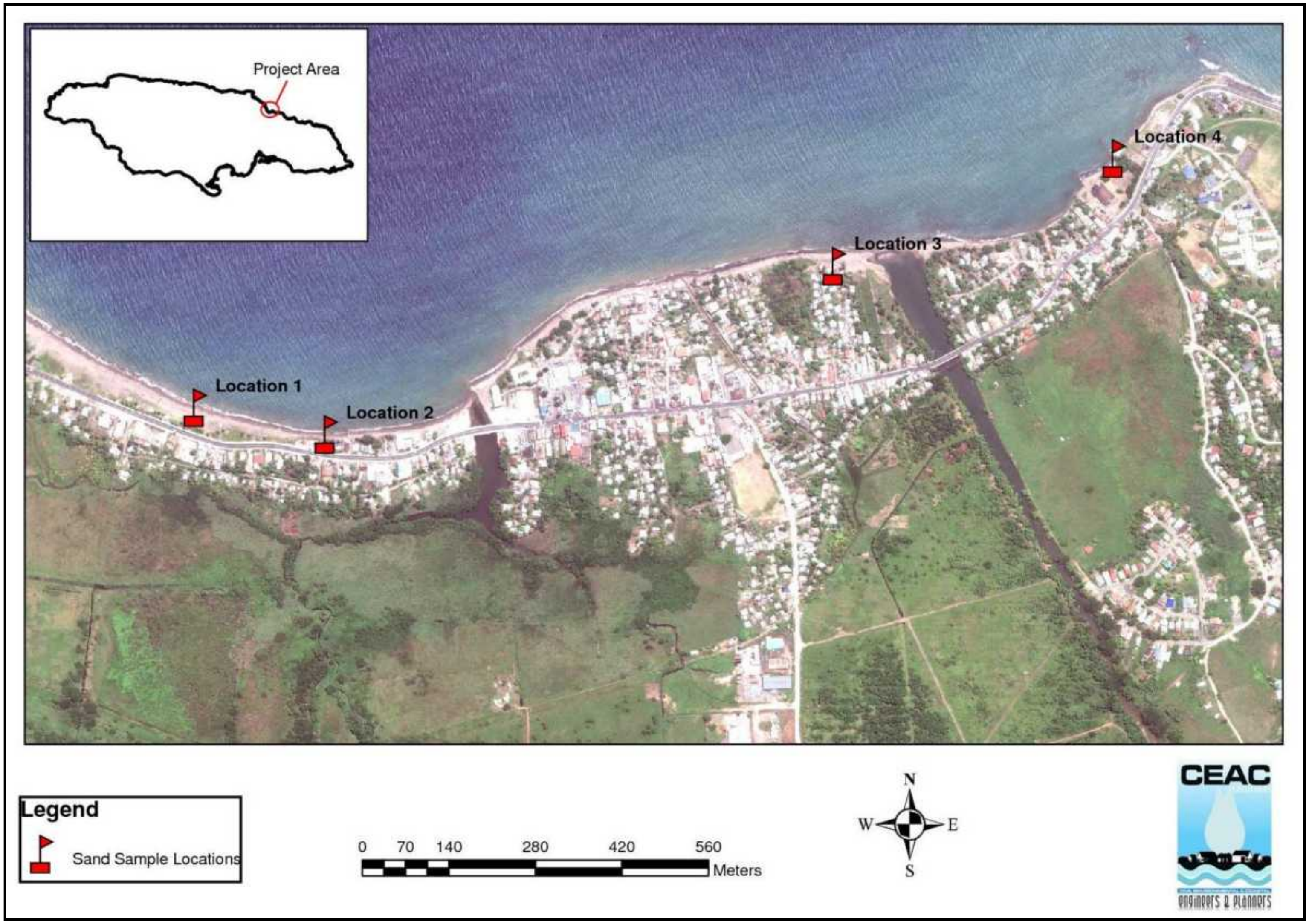


Figure 0.5 Sediment sample locations

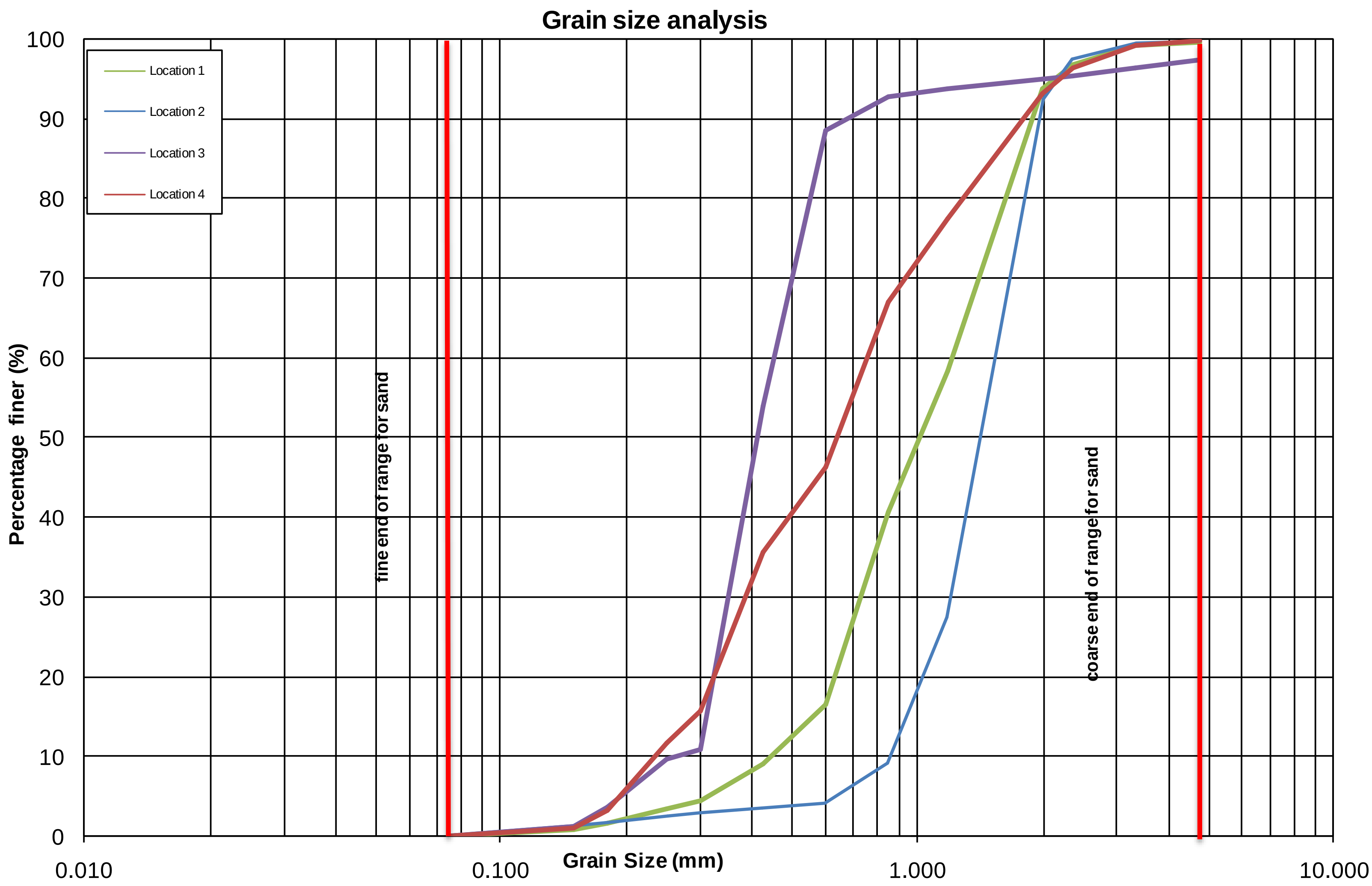


Figure 0.6 Results of Sieve Analysis conducted for project area

Storm Surge Hazard

Methodology

It was necessary to define the deepwater hurricane wave climate at a point offshore Port Royal

- Latitude: 18.4590 degrees North
- Longitude: -76.670 degrees West

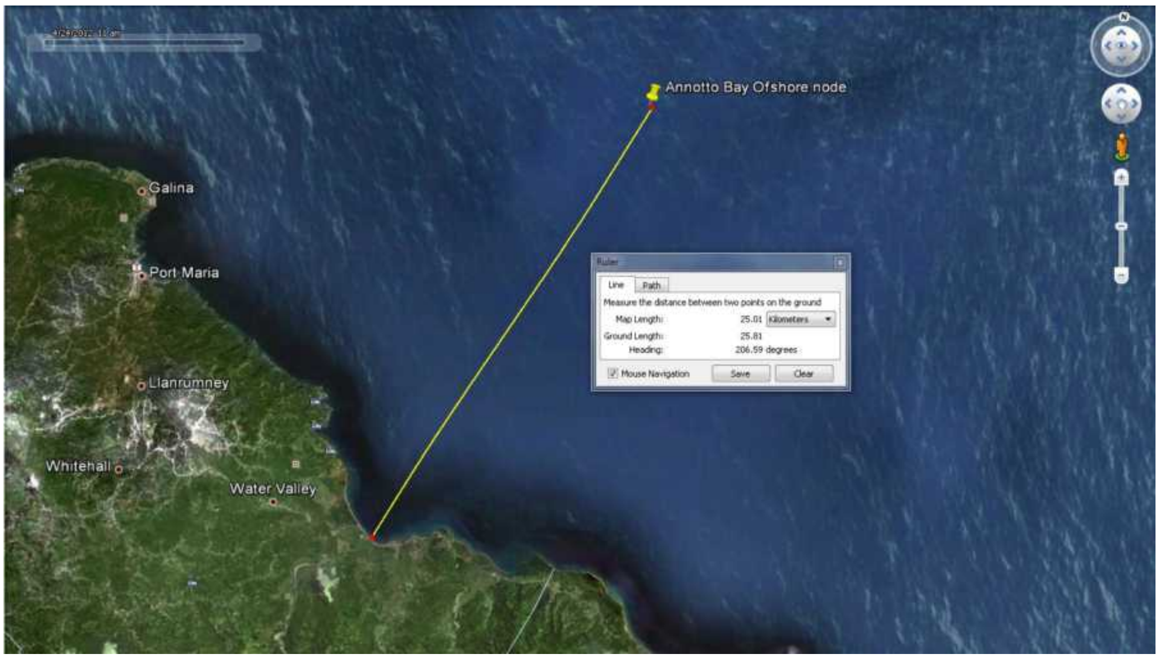


Figure 0.7 - Location of offshore point used for Extremal analysis, showing the track used in the analysis

The National Hurricane Center (NOAA) database of hurricane track data in the Caribbean Sea was utilized to carry out a hindcast, followed by a statistical analysis to determine the hurricane: waves, wind and set-up conditions

The database of hurricanes, dating back to 1886, was searched for storms that passed within a 300km radius from the site. The following procedure was carried out.

1. Extraction of Storms and Storm Parameters from the historical database. A historical database of storms was searched for all storms passing within a search radius of 300km radius of the site.

2. Application of the JONSWAP Wind-Wave Model. A wave model was used to determine the wave conditions generated at the site due to the rotating hurricane wind field. This is a widely applied model and has been used for numerous engineering problems. The model computes the wave height from a parametric formulation of the hurricane wind field.
3. Application of Extremal Statistics. Here the predicted maximum wave height from each hurricane was arranged in descending order and each assigned an exceedence probability by Weibull's distribution.
4. A bathymetric profile from deepwater to the site was then defined and each hurricane wave transformed along the profile. The wave height at the nearshore end of the profile was then extracted from the model and stored in a database. All the returned nearshore values were then subjected to an Extremal Statistical analysis and assigned exceedance probabilities with a Weibull distribution.

Results

Occurrences and Directions

The results of the search from the database for hurricanes that came within the search radius of the site are shown in Table 3.1. Extremal analysis results are summarized in the bi-variant Table 3.2. The results of the search clearly indicate the sites overall vulnerability to such systems. In summary:

- 78 hurricane systems came within 300 kilometers of the project area
- 2 of which were classified as catastrophic (Category 5)
- 18 were classified as extreme (Category 4)

The bi-variant table analysis indicates that the waves generated offshore the site have approached from all seaward possible. However, the most frequent hurricane waves have been noted to come from an **easterly** direction. In summary, there are:

- **46 (x6 hours) occurrences from the East**
- 45 (x6 hours) occurrences from the North-West
- 41 (x6 hours) occurrences from the North-East
- 23 (x6 hours) occurrences from the North
- 19 (x6 hours) occurrences from the West

The east, north-west and north-east directions are more prevalent for the node considered because of the seaward projection of the eastern part of the island that some what buffer the site from remote easterly waves. The site however becomes more exposed as soon as the passing hurricane systems are more to the north-east of the island.

Table 0.2 – Name of storm events that passed within 300 km of Annotto Bay since 1851.

	Storm No.	Name	Date	Max. SS Category			Storm No.	Name	Date	Max. SS Category			Storm No.	Name	Date	Max. SS Category	
1	4	NOTNAMED	1851	3-	EXTENSIVE	31	549	NOTNAMED	1924	1-	WEAK	61	1314	LILI	2002	4-	EXTREME
2	51	NOTNAMED	1859	3-	EXTENSIVE	32	574	NOTNAMED	1928	2-	MODERATE	62	1349	DENNIS	2005	4-	EXTREME
3	92	NOTNAMED	1865	3-	EXTENSIVE	33	585	NOTNAMED	1930	4-	EXTREME	63	1379	ERNESTO	2006	1-	WEAK
4	131	NOTNAMED	1870	3-	EXTENSIVE	34	592	NOTNAMED	1931	2-	MODERATE	64	1398	OLGA	2007	1-	WEAK
5	154	NOTNAMED	1873	3-	EXTENSIVE	35	602	NOTNAMED	1932	3-	EXTENSIVE	65	1404	FAY	2008	1-	WEAK
6	161	NOTNAMED	1874	2-	MODERATE	36	609	NOTNAMED	1933	1-	WEAK	66	1405	GUSTAV	2008	4-	EXTREME
7	164	NOTNAMED	1875	3-	EXTENSIVE	37	625	NOTNAMED	1933	2-	MODERATE	67	1414	PALOMA	2008	4-	EXTREME
8	185	NOTNAMED	1878	2-	MODERATE	38	637	NOTNAMED	1934	1-	WEAK	68	1276	DEBBY	2000	1-	WEAK
9	203	NOTNAMED	1880	1-	WEAK	39	642	NOTNAMED	1935	3-	EXTENSIVE	69	1280	HELENE	2000	1-	WEAK
10	232	NOTNAMED	1884	2-	MODERATE	40	643	NOTNAMED	1935	1-	WEAK	70	1311	ISIDORE	2002	3-	EXTENSIVE
11	245	NOTNAMED	1886	4-	EXTREME	41	682	NOTNAMED	1939	2-	MODERATE	71	1314	LILI	2002	4-	EXTREME
12	246	NOTNAMED	1886	3-	EXTENSIVE	42	705	NOTNAMED	1942	1-	WEAK	72	1332	BONNIE	2004	1-	WEAK
13	254	NOTNAMED	1887	1-	WEAK	43	720	NOTNAMED	1944	3-	EXTENSIVE	73	1349	DENNIS	2005	4-	EXTREME
14	257	NOTNAMED	1887	1-	WEAK	44	773	NOTNAMED	1949	2-	MODERATE	74	1379	ERNESTO	2006	1-	WEAK
15	265	NOTNAMED	1887	1-	WEAK	45	786	KING	1950	3-	EXTENSIVE	75	1398	OLGA	2007	1-	WEAK
16	328	NOTNAMED	1894	3-	EXTENSIVE	46	828	HAZEL	1954	4-	EXTREME	76	1404	FAY	2008	1-	WEAK
17	361	NOTNAMED	1898	1-	WEAK	47	863	ELLA	1958	3-	EXTENSIVE	77	1405	GUSTAV	2008	4-	EXTREME
18	370	NOTNAMED	1899	1-	WEAK	48	865	GERDA	1958	1-	WEAK	78	1414	PALOMA	2008	4-	EXTREME
19	371	NOTNAMED	1900	4-	EXTREME	49	894	GERDA	1961	1-	WEAK						
20	379	NOTNAMED	1901	1-	WEAK	50	909	FLORA	1963	4-	EXTREME						
21	384	NOTNAMED	1901	1-	WEAK	51	916	CLEO	1964	4-	EXTREME						
22	405	NOTNAMED	1904	1-	WEAK	52	938	INEZ	1966	4-	EXTREME						
23	413	NOTNAMED	1905	3-	EXTENSIVE	53	942	BEULAH	1967	5-	CATASTROPHIC						
24	438	NOTNAMED	1908	2-	MODERATE	54	1028	ELOISE	1975	3-	EXTENSIVE						
25	451	NOTNAMED	1909	2-	MODERATE	55	1063	CLAUDETTE	1979	1-	WEAK						
26	454	NOTNAMED	1910	2-	MODERATE	56	1070	ALLEN	1980	5-	CATASTROPHIC						
27	469	NOTNAMED	1912	3-	EXTENSIVE	57	1206	GORDON	1994	1-	WEAK						
28	478	NOTNAMED	1915	4-	EXTREME	58	1276	DEBBY	2000	1-	WEAK						
29	488	NOTNAMED	1916	4-	EXTREME	59	1280	HELENE	2000	1-	WEAK						
30	501	NOTNAMED	1917	4-	EXTREME	60	1311	ISIDORE	2002	3-	EXTENSIVE						

Table 0.3 – Bivariant table of Extremal wave climate for Annotto Bay

Wind direction- NW

Tp(s)	Wave height(m)										Total
<value	2	4	6	8	10	12	14	16	18	20	
2											
4											
6		4									4
8		23	9								32
10			8	1							9
12											
14											
16											
18											
20											
Total		27	17	1							45

Wind direction- N

Tp(s)	Wave height(m)										Total
<value	2	4	6	8	10	12	14	16	18	20	
2											
4											
6		10									10
8		9	4								13
10											
12											
14											
16											
18											
20											
Total		19	4								23

Wind direction- NE

Tp(s)	Wave height(m)										Total
<value	2	4	6	8	10	12	14	16	18	20	
2											
4											
6		5									5
8		9	13								22
10			13	1							14
12											
14											
16											
18											
20											
Total		14	26	1							41

Wind direction- W

Tp(s)	Wave height(m)										Total
<value	2	4	6	8	10	12	14	16	18	20	
2											
4											
6		2									2
8		7	5								12
10			5								5
12											
14											
16											
18											
20											
Total		9	10								19

All directions

Tp(s)	Wave height(m)										Total
<value	2	4	6	8	10	12	14	16	18	20	
2											
4											
6		25	2								27
8		64	41								105
10			43	6							49
12											
14											
16											
18											
20											
Total		89	86	6							181

Wind direction- E

Tp(s)	Wave height(m)										Total
<value	2	4	6	8	10	12	14	16	18	20	
2											
4											
6		2	2								4
8		16	7								23
10			16	3							19
12											
14											
16											
18											
20											
Total		18	25	3							46

Wind direction- SW

Tp(s)	Wave height(m)										Total
<value	2	4	6	8	10	12	14	16	18	20	
2											
4											
6											
8											
10											
12											
14											
16											
18											
20											
Total											

Wind direction- S

Tp(s)	Wave height(m)										Total
<value	2	4	6	8	10	12	14	16	18	20	
2											
4											
6											
8											
10											
12											
14											
16											
18											
20											
Total											

Wind direction- SE

Tp(s)	Wave height(m)										Total
<value	2	4	6	8	10	12	14	16	18	20	
2											
4											
6		2									2
8			3								3
10			1	1							2
12											
14											
16											
18											
20											
Total		2	4	1							7

Storm surge

Static storm surge was investigated in the analysis for all major components of storm surge. The phenomena considered were:

- Wave breaking and shoaling
- Wind set-up
- Refraction
- Tides
- Global Sea Level Rise (over a 50 year project life)
- Inverse Barometric Pressure Rise

The eastern profile is the most extreme as shown in Figure 3.2. See Table 3.3. The results indicate that the expected 100 Year storm surge is 1.88 metres.

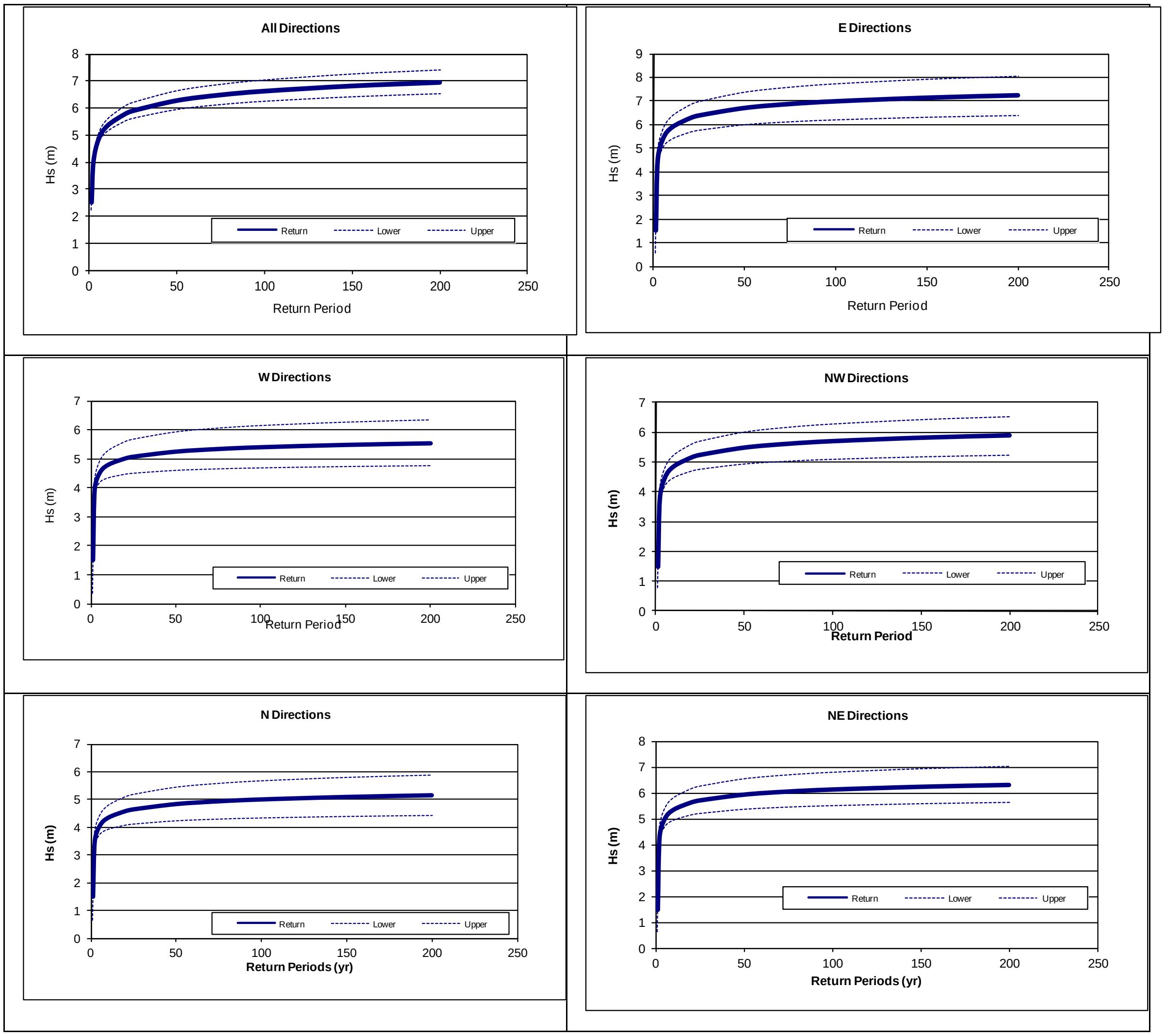
Table 0.4 – Extremal Storm surge (metres) predictions for the site along the profile from shoreline to deepwater for all directional waves possible for Annotto Bay

Return Period	Total setup (m)								
	All	SW	W	NW	N	NE	E	SE	S
1		0.00	0.00	NaN	0.00	NaN	NaN	0.00	0.00
2	NaN	0.00	0.00	NaN	0.00	NaN	0.55	0.00	0.00
5	0.71	0.00	0.00	0.60	0.00	0.69	0.95	0.00	0.00
10	0.88	0.00	0.00	0.70	0.00	0.80	1.19	0.00	0.00
20	1.05	0.00	0.00	0.80	0.00	0.91	1.41	0.00	0.00
25	1.10	0.00	0.00	0.83	0.00	0.94	1.48	0.00	0.00
50	1.27	0.00	0.00	0.92	0.00	1.05	1.68	0.00	0.00
75	1.37	0.00	0.00	0.97	0.00	1.11	1.80	0.00	0.00
100	1.44	0.00	0.00	1.01	0.00	1.15	1.88	0.00	0.00
150	1.54	0.00	0.00	1.06	0.00	1.21	2.00	0.00	0.00
200	1.61	0.00	0.00	1.09	0.00	1.26	2.08	0.00	0.00

Confidence levels

The maximum and minimum confidence limits showed increased variance from the return value as the return period increases. The confidence limits for the setups showed an average variance of less than 0.36m between return value and the maximum and minimum levels for the 100 year return period. This is reasonable given that the source data covers 125 years.

Table 0.5 Deepwater Wave heigths (Hs) and confidence limits for the different westerly to easterly directions



Anecdotal Evidence of Storm Surge

Anecdotal evidence of past storms was collected to aid in the verification of a storm surge model for the area. Such evidence was also used to generate an estimate of the return period for actual storm surge versus estimated.

Interviews were conducted with available residents of Annotto Bay with living first hand memory of hurricane events. Overall, twenty effective interviews were done with residents with an average age of 52.2 years and living an average of 41 years in Annotto Bay. The respondents recalled 5 storms, including: Allen (1980), Gilbert (1988), Ivan (2004), Dennis (2005) and Gustav (2008). The resulting average setup for each storm is summarized in Table 4.6 below and compared to the model predicted setup. See also Figure 4.3 below.

Table 0.6 Summary of anecdotal information collected in Annotto Bay, July 2012.

ID	Name of Interviewee	Age (years)	Time in Area (years)	Storm	Year	Locations		Storm Surge			Comments
						Interview location	Debris on Road from Wave Run up	Depth of water (m)	Ground elevn (m)	Water Elevation (m)	
1	Collette Paul	45	45	Ivan	2004	342	plastic bottles; garbage	0.300	1.113	1.413	water runs up through community lane; wave runup
2a	Delroy Gyles	42	42	Gustav	2008	344	tree limbs; coconuts	0.300	1.125	1.425	
2b	Delroy Gyles	42	42	Dennis	2005	344	tree limbs; coconuts	0.500	1.125	1.625	whole lane flooded; water reaches main road
3a	Esmeralda Bleacher	60	50	Allen	1980	346	bamboo; coconut; plastics	0.250	0.990	1.240	river discharges into sea but is pushed back on land due to waves
3b	Esmeralda Bleacher	60	50	Gustav	2008	346	bamboo; coconut; plastics	0.300	0.990	1.290	tree fell on house
4	Rudoff Brown	60	60	Gustav	2008	347	tree branches; coconuts	0.500	0.829	1.329	house invaded by water (sheet flow only)
5	Clovis Barnett	27	17	Ivan	2004	349	seaweed; seagrass	0.300	1.700	2.000	church yard flooded; water never reached road
6	Carnel Downer	24	24	Dennis	2005	355	trees; fish; plastics	0.500	2.700	3.200	police evacuates residents
7a	Alexander Parker	58	9	Ivan	2004	356	seaweed; bamboo	1.200	2.350	3.550	Water run up reaches the church boundary road
7b	Alexander Parker	58	9	Gustav	2008	356	plastics; garbage	0.900	2.350	3.250	furniture observed floating out of house
8	Glen Appleton	32	32	Dennis	2005	358	bamboo; guango; coconut; plastics	0.900	1.700	2.600	swamp across road overflows and add to storm surge
9	Winston Brown	76	76	Allen	1980	361	stone; plastics; tree limbs	0.400	0.340	0.740	water reaches main road; water levels high enough for boats
10	Fay Brown	58	48	Dean	2007	362	garbage; wood; coconut	0.400	0.227	0.627	house adjacent to beach invaded by water
11	Escoffery Jackson	56	56	Gustav	2008	364	banana; coconut; rocks	1.000	0.286	1.286	house adjacent to beach invaded by water and sand
12	Sonia McDonald	57	57	Allen	1980	365	coconut; bamboo; plastics	0.300	0.639	0.939	plyboard on beachfront shop destroyed by sea
13	Patrick Francis	71	35	Allen	1980	366	stones; plastics	0.600	0.914	1.514	gabion baskets destroyed
14	Wayne Bennett	42	42	Ivan	2004	367	plastics; tree limbs	1.200	0.255	1.455	house on beachfront destroyed (had to be rebuilt)
15	Leon Rogers	39	39	Gilbert	1988	368	stones; sand; plastics	1.200	1.041	2.241	--
16	Natasha Ricketts	36	28	Dean	2007	369	tree limbs; stones	0.800	0.864	1.664	--
17	Bibsy Gardener	61	61	Ivan	2004	378	tree limbs; bamboo	0.400	1.850	2.250	water gets contained between banking and main road

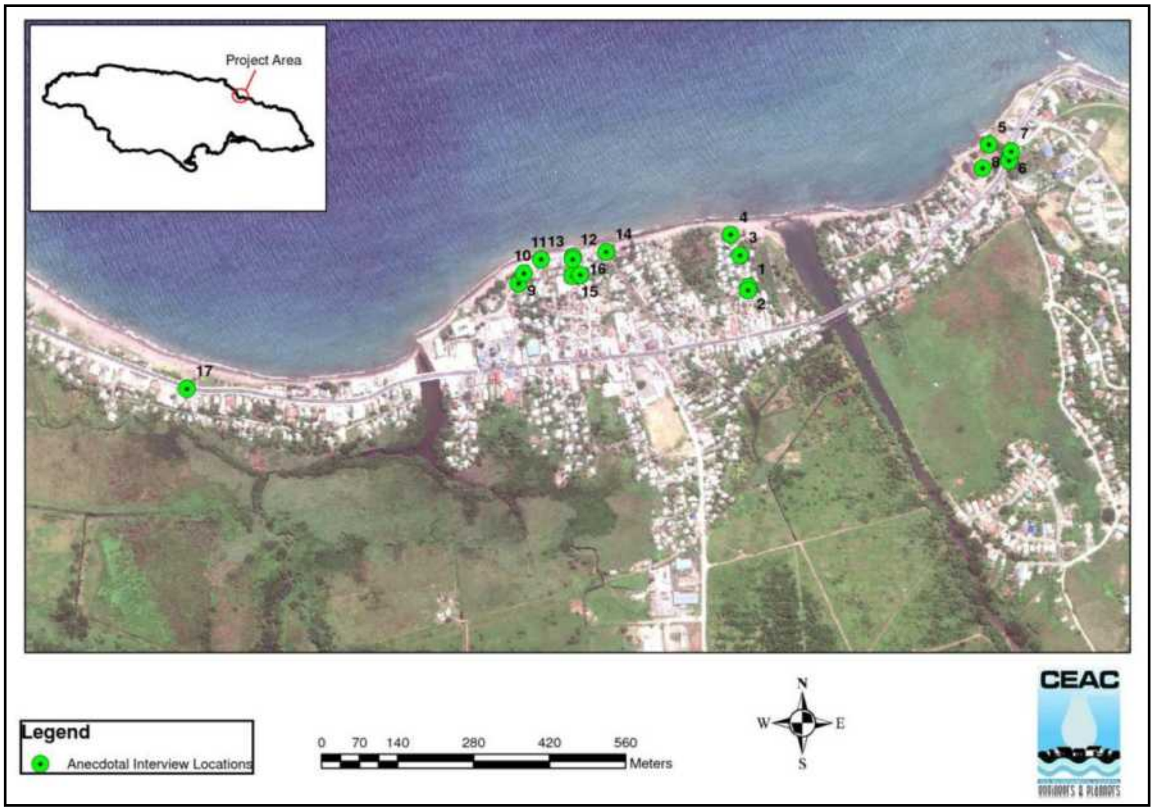


Figure 0.8 Locations in Annotto Bay where anecdotal interviews were conducted.

Table 0.7 Observed average setup (based on interviews) versus model predicted setup

Hurricane	Oberved-Average setup (m)	Predicted-Average setup (m)	Difference (m)
Allen	1.108	1.3798	-0.272
Gustav	1.716	0.9666	0.749
Dennis	2.113	1.0486	1.064
		Average	**0.514**

Two storms were eliminated from the average observed setups and comparisons to model results, as they conflicted with what the general understanding of what should have occurred. Ivan passed on the southern coast of Jamaica and could not have generated significant storm surge in Annotto Bay as some respondents had reported. Hurricane Gilbert passed on over the island and was also eliminated for the same reason.

Notwithstanding, the model results for hurricane Allen compares well with the model being slightly higher by 0.27m, whereas Gustav and Dennis were higer by 0.77 and 1.46m respectively.

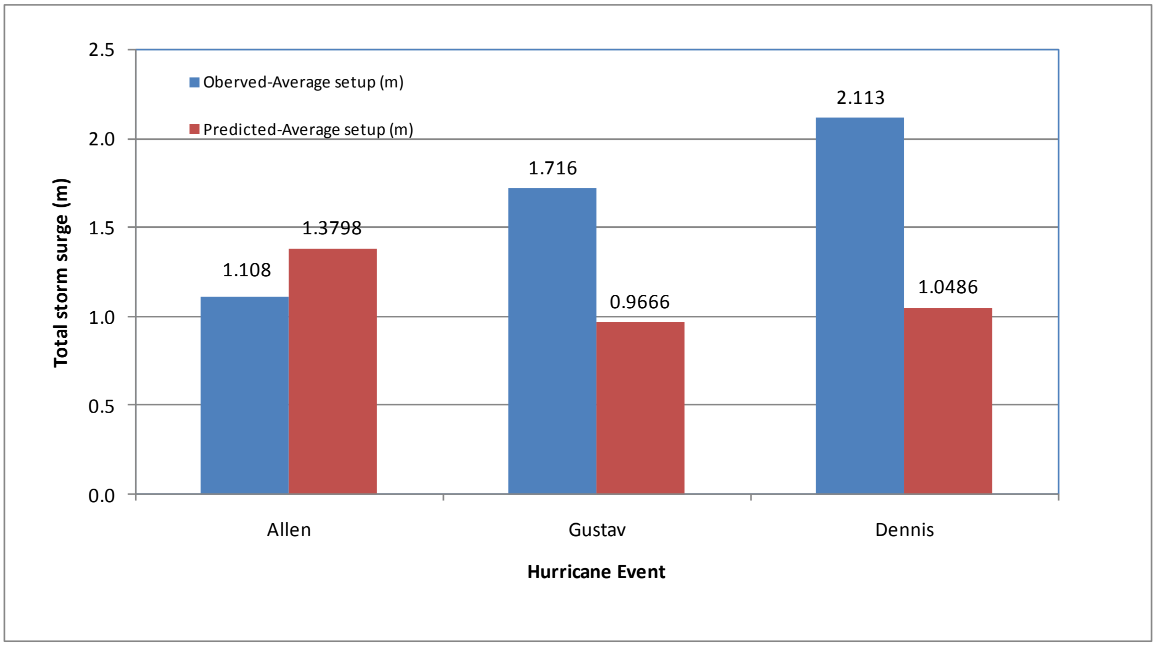


Figure 0.9 Observed average setup (based on interviews) versus model predicted setup

The observed setups were subjected to extremal statistical analysis to estimate the return period of the setups experienced. The statistical tool used was the Weibull function which is widely used for this type of extremal data analysis due to it having three variables which enables it to obtain a better fitted curve those others which have only two variables.

One factor that was unaccounted for in the model prediction however is the effect of wave run-up which will inevitably increase the water levels. This parameter would not have been easily differentiable to the observers and would have thus been a part of what was observed. It is against this background that wave run-up was determined and added to the storm surge elevations.

The Software programme Cresswin was utilized to estimate the runup. This software uses the model for wave run-up on smooth and rock slopes of coastal structures according to Van der Meer et al[7].

The Estimated run-up ranges from 0.705m to 1.441m for the 5 to 100 year hurricane waves and were added to the model predicted storm surge results.

[7] J.W. Van Deer Meer and C.J.M. Stam(1992), Wave run-up on smooth and rock slopes of coastal structures by, ASCE journal of WPC&OE, vol 118, pp534 – 550

Table 0.8 Summary of predicted storm surge based on observations and CEAC model predictions for different return periods.

Ret. Period	Predicted storm surge-from observations (m)	Predicted storm surge-from model without run-up(m)	Predicted storm surge-from model with run-up(m)
2	0.05		
5	0.33	0.705	1.355
10	0.74	0.876	1.606
25	1.58	1.102	1.892
50	2.47	1.272	2.092
100	3.58	1.441	2.321

The CEAC model predictions with run-up are less intense than the reported trends in Annotto Bay over the past 32. The trend lines for the model predicted and the reported surge levels differ in shape with the observed trends being much steeper then the model predicted.

On the other hand, the model predicted and runup trend lines were similar in shape, indicating that if there is and error in the CEAC models it is constant and predictable. This scenario would be more reliable that one where the error is variable.

Given the inconsistencies in the reporting of the storm surge levels and the uncharacteristically sharp increases in surges for the higher return periods, the CEAC model with run-up was therefore chosen as the benchmark model for use in determining the 10, 50 and 100yr return period storm surge levels for Annotto Bay.

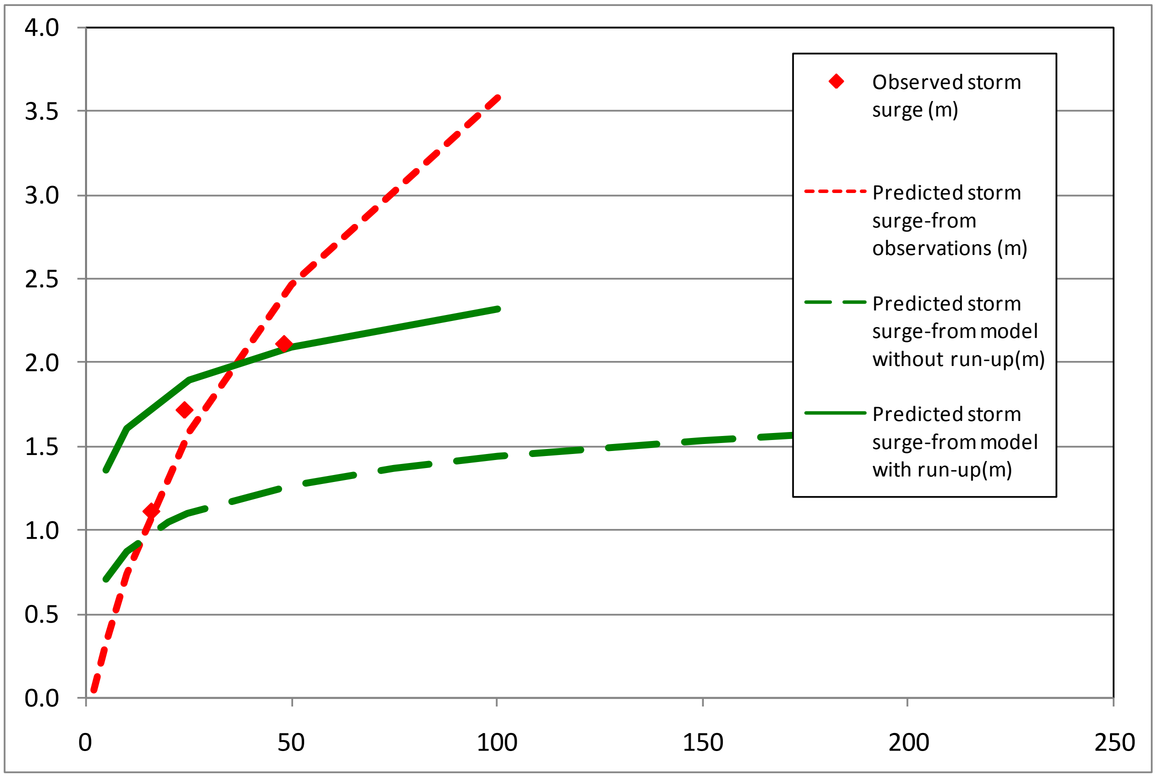


Figure 0.10 Plots of predicted and observed storm surge elevations over different return periods

Storm Surge Inundation Levels

The flood levels for the 10, 50 and 100yr flood level contours were plotted over the digital terrain model of the project site. This terrain is based on survey data gathered by the CEAC team during site visits, and supplemented by the National land Survey Departments' 12500 map series data. Figure 3.5 below shows that most of the developments to the north of the highway are susceptible to flooding as a result of the 1in10 to 1in100 year storm surge. The maximum extents of flooding was estimated from the shoreline at 312m, 378m, and 407m inland for the 10, 50 and 100 year return storm surges.

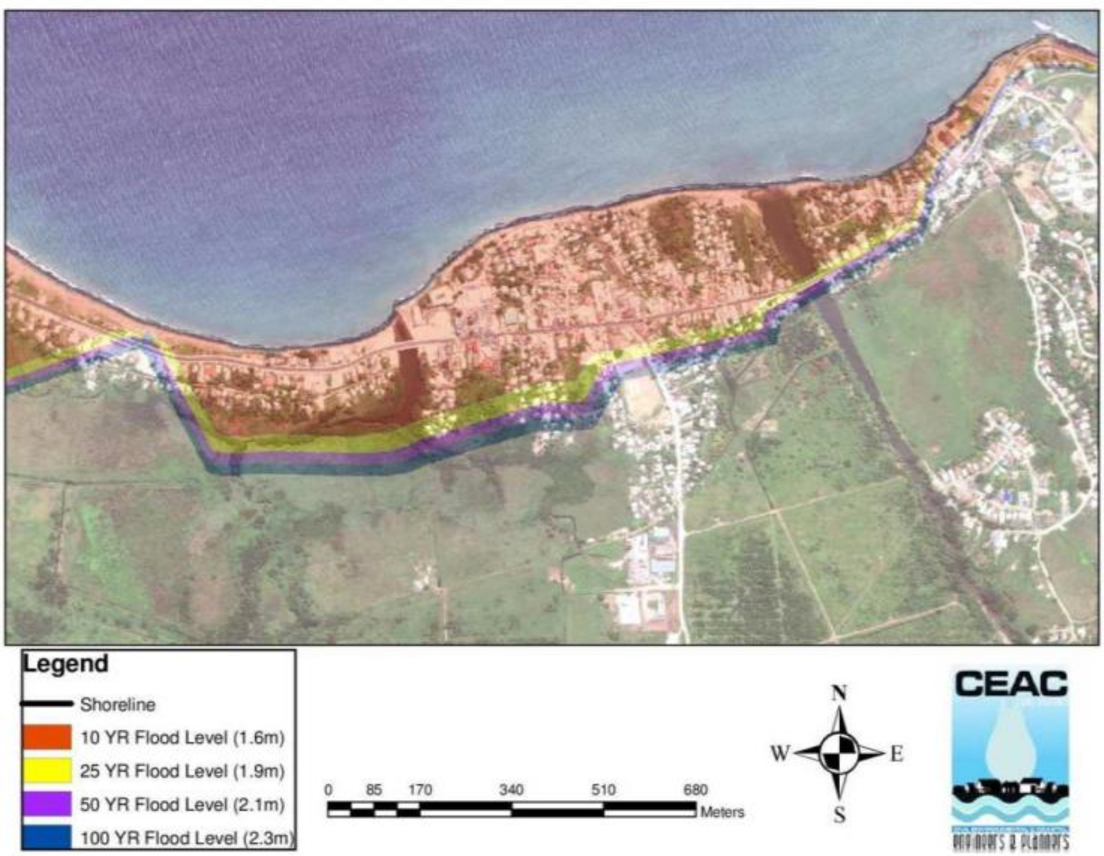


Figure 0.11 10, 25, 50 and 100 year return periods flood levels.

Shoreline Erosion Hazard

Deep Hurricane Water Wave Climate

It was necessary to define the deepwater hurricane wave climate at a point offshore Port Royal

- Latitude: 17.760 degrees North
- Longitude: 76.790 degrees West

Method

The National Hurricane Center (NOAA) database of hurricane track data in the Caribbean Sea was utilized to carry out a hindcast, followed by a statistical analysis to determine the hurricane: waves, wind and set-up conditions

The database of hurricanes, dating back to 1886, was searched for storms that passed within a 500km radius from the site. The following procedure was carried out.

1. Extraction of Storms and Storm Parameters from the historical database. A historical database of storms was searched for all storms passing within a search radius of 300km radius of the site.
2. Application of the JONSWAP Wind-Wave Model. A wave model was used to determine the wave conditions generated at the site due to the rotating hurricane wind field. This is a widely applied model and has been used for numerous engineering problems. The model computes the wave height from a parametric formulation of the hurricane wind field.
3. Application of Extremal Statistics. Here the predicted maximum wave height from each hurricane was arranged in descending order and each assigned an exceedance probability by Weibull's distribution.

All the returned values were then subjected to an Extremal Statistical analysis and assigned exceedance probabilities with a Weibull distribution.

Results

The hurricane waves originating from the East are the most severe of all the directions investigated, followed closely by the North-easterly winds. See Table 4.1

below. The eastern waves are however not expected to significantly impact the site as much the North-easterly profile would, due to the Northern projection of the land by Iter Boreale, which is located east of Annotto Bay.

Table 0.9 Summary of wave heights and periods from various directions for different return periods

Return Periods	Wave height (m)																	
	All		SW		W		NW		N		NE		E		SE		S	
	Hs	Tp	Hs	Tp	Hs	Tp	Hs	Tp	Hs	Tp	Hs	Tp	Hs	Tp	Hs	Tp	Hs	Tp
1	2.5	8.0	0.0	0.0	1.5	6.2	1.5	6.2	1.5	6.2	1.5	6.2	1.5	6.2	0.0	0.0	0.0	0.0
2	4.0	10.0	0.0	0.0	4.0	10.0	3.8	9.8	3.5	9.4	4.3	10.4	4.5	10.6	0.0	0.0	0.0	0.0
5	4.8	11.0	0.0	0.0	4.5	10.6	4.5	10.6	4.1	10.1	5.0	11.2	5.4	11.6	0.0	0.0	0.0	0.0
10	5.3	11.5	0.0	0.0	4.8	10.9	4.9	11.0	4.3	10.4	5.4	11.5	5.9	12.1	0.0	0.0	0.0	0.0
20	5.8	12.0	0.0	0.0	5.0	11.2	5.1	11.3	4.6	10.7	5.6	11.8	6.3	12.4	0.0	0.0	0.0	0.0
25	5.9	12.1	0.0	0.0	5.1	11.2	5.2	11.4	4.6	10.8	5.7	11.9	6.4	12.5	0.0	0.0	0.0	0.0
50	6.3	12.5	0.0	0.0	5.2	11.4	5.5	11.7	4.8	11.0	5.9	12.1	6.7	12.8	0.0	0.0	0.0	0.0
75	6.5	12.7	0.0	0.0	5.3	11.5	5.6	11.8	4.9	11.1	6.1	12.2	6.9	13.0	0.0	0.0	0.0	0.0
100	6.6	12.8	0.0	0.0	5.4	11.6	5.7	11.9	5.0	11.2	6.1	12.3	7.0	13.1	0.0	0.0	0.0	0.0
150	6.8	13.0	0.0	0.0	5.5	11.7	5.8	12.0	5.1	11.3	6.2	12.4	7.1	13.2	0.0	0.0	0.0	0.0
200	7.0	13.1	0.0	0.0	5.5	11.7	5.9	12.1	5.1	11.3	6.3	12.5	7.2	13.3	0.0	0.0	0.0	0.0

Nearshore Wave Climate and Shoreline Erosion

Model Description

SBEACH is an empirically based numerical model for estimating beach and dune erosion due to storm waves and water levels. The magnitude of cross-shore sand transport is related to wave energy dissipation per unit water volume in the main portion of the surf zone. The direction of transport is dependent on deep water wave steepness and sediment fall speed. SBEACH is a short-term storm processes model and is intended for the estimation of beach profile response to storm events. Typical simulation durations are limited to hours to days (1 week maximum).

Model Input

Profiles were cut from deepwater to land up to a maximum elevation of approximately six metres at four locations from northern and north easterly directions spanning the entire project shoreline. The wave data from the deep water hurricane model were utilized for this analysis to represent the most vulnerable directions. Table 4.2 shows the 100 year return period wave characteristics utilized in the model and the input parameters for the model for each profile. Other input parameters included the sediment grainsize on the beach face and storm duration.

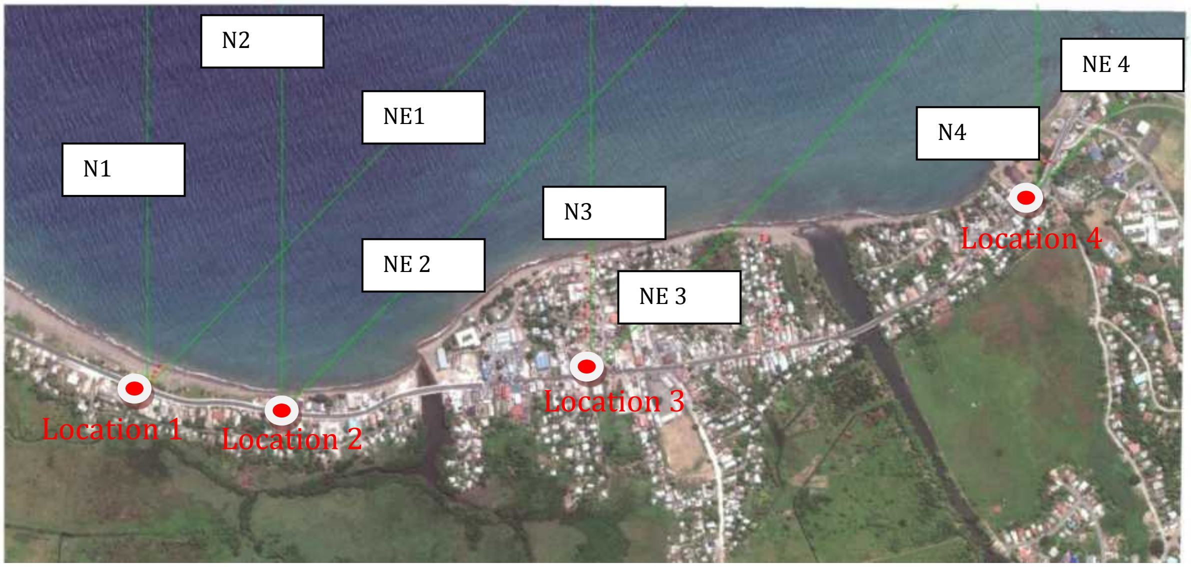


Figure 0.12 Locations along shoreline where profiles were cut relative to the project site

Table 0.10 SBEACH input parameters for 10, 50 and 100 year return storm

Locations	10	50	100
Directions	All	All	All
Input Parameters			
Hs (m)	5.80	6.68	6.90
Tp (s)	12.1	12.84	13.10
Deepwater storm Surge	0.88	1.27	1.44

Results

Nearshore wave height

The maximum wave heights estimated at the shoreline as a result of wave transformation varies from 1.8m to 3.4m from the 10 to 100 year storm. See Table 0.11 below. These wave heights arriving at the shoreline possess the potential for serious damage to the beach and to structures behind the beach. Due consideration should therefore be given to building potential protective structures in this area.

Table 0.11 Nearshore wave heights for 1in10 to 1in100 year return storm for Annotto Bay

Storm	Average maximum Wave height at shoreline for all directions (m)
10yr	1.8
50yr	3.3
100yr	3.4

Erosion

The erosion vulnerability of the shoreline to the four locations along each profile is summarized in Table 4.3. All the profiles in Table 4.3 were plotted over the project area and relative to the shoreline to obtain the actual setbacks from the shoreline. Overall the shoreline is predicted to erode between 24 to 38 metres during a 10 year storm from the north eastern and northerly directions in locations one and three respectively. See Table 0.12 below. No erosion was predicted for the higher return periods. This is because the wave heights arriving at the shoreline are not significantly different, but the differences in the setups are much larger. This has resulted in the waves exerting more force on the shoreline and causing erosion.

Table 0.12 Estimated erosion distances (metres) along each profile for the 10 yr storm for Annotto Bay

Location	Profile	Shoreline	Farthest erosion Location on Profile	Erosion (m)
1	N	45	45	0
	NE	49	11	38
2	N	30	30	0
	NE	44	44	0
3	N	210	186	24
	NE	371	371	0
4	N	118	118	0
	NE	285	285	0

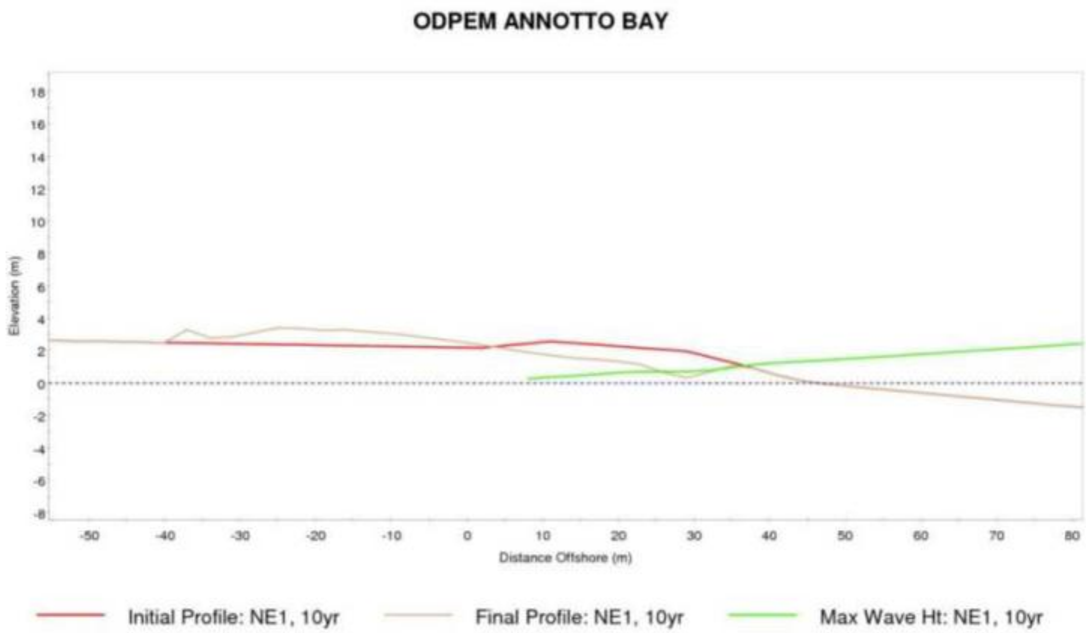


Figure 0.13 NE 1 profile for 10yr storm at location 1 showing the predicted erosion

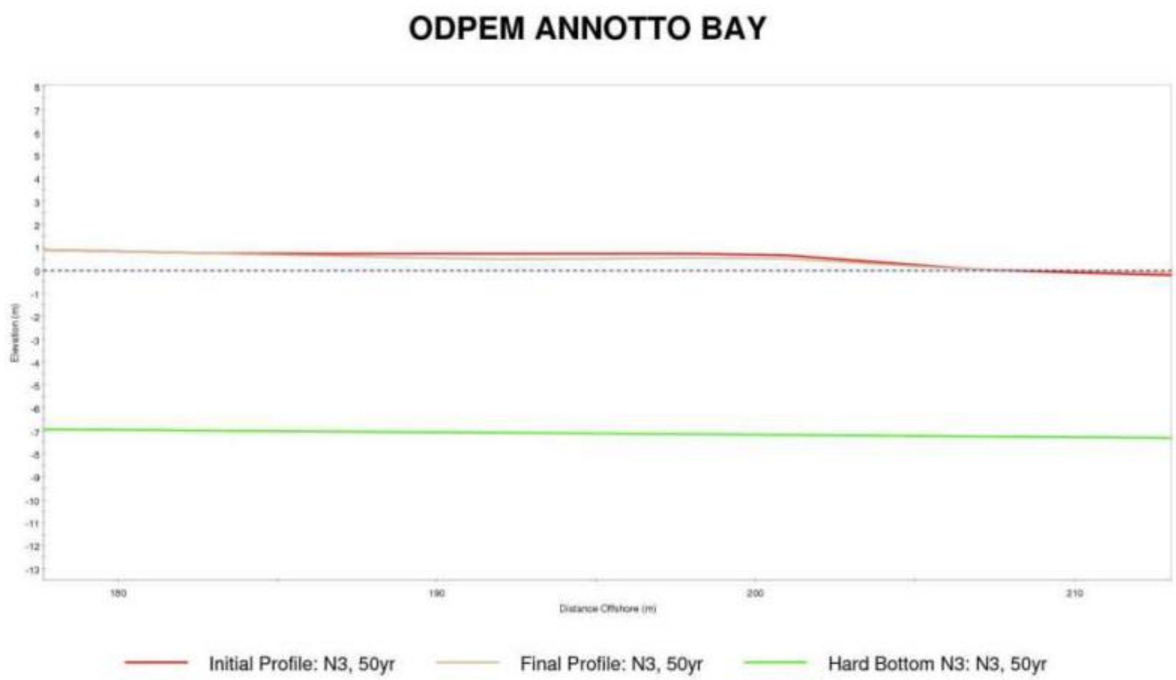


Figure 0.14 N3 profile for 10yr storm at location 1 showing the predicted storm surge and erosion

Figure 0.15 Erosion map of Annotto shoreline for 10 yr storm

Mitigation Measures

Several mitigation measures are applicable to the flooding, storm surge and erosion hazards of Annatto Bay. These include:

1. Groin for drains and gully to keep the discharge to the sea clear
2. Shoreline protection in the form of buried revetments for the vulnerable areas
3. Building permitting enforcements of higher floor levels to minimize the risks of storm surge

The cost of these options are roughly estimated to be about USD 4.0 Million. A specific solution for the Motherford drain was conceptual designed herein as shown in Figure 6.1. It is estimated to 136 metres long and 3 metres wide at the crest. The estimated cost is USD 1,033,692.

Figure 0.16 Motherford Darin Groin Design, Annotto Bay, St. Mary

Engineering Cost Estimate

Table 0.13 Estimate for groin construction

ODPEM Annotto Bay Groin Construction					
NR	**DESCRIPTION**	**UNIT**	**QUANTITY**	**RATE**	**Amount**
1.00	**PRELIMINARIES**	Sum	1	$ 25,000.00	$ 25,000.00
1.10	**EARTHWORKS**	Sum	1	$ 25,000.00	$ 25,000.00
1.20	**FEES - Design & Detailing**	Sum	1	$ 15,000.00	$ 15,000.00
1.30	**FEES - Monitoring**	Sum	1	$ 25,000.00	$ 25,000.00
	SUB-TOTAL				**$ 90,000.00**
NR	**DESCRIPTION**	**UNIT**	**QUANTITY**	**RATE**	
2.00	**REVETMENT**				
2.01	Cleaning and Grubbing to remove top 150mm of topsoil from existing subgrade within work area and store within 300 metres of site.	M3	450	$ 5.00	$ 2,250.00
2.02	Excavate for toe	M3	2680	$ 12.00	$ 32,160.00
2.03	Geotextile	M3	2680	$ 20.00	$ 53,600.00
2.04	Armour to revetment (1.5 M crest elevation)	M3	4020.0	$ 170.00	$ 683,400.00
2.05					
2.06					
	SUB-TOTAL				**$ 771,410.00**
	SUMMARY				
	PRELIMINARIES				$ 25,000.00
	EARTHWORKS				$ 25,000.00
	FEES - Design & Detailing				$ 15,000.00
	FEES - Monitoring				$ 25,000.00
	REVETMENT				$ 771,410.00
	TOTAL				$ 861,410.00
	CONTINGENCIES (20%)				$ 172,282.00
	GRAND TOTAL				$ 1,033,692.00

Conclusions and Recommendations

Conclusions

The following conclusions were made drawn based on the data collection and analuyysis conducted to date:

1. Historical and anecdotal evidence revealed Annotto Bay is vulnerable to flooding from storm surges as a result of the hurricanes. Wave transformation modelling indicates the maximum wave heights estimated at the shoreline as a result of wave transformation varies from 1.8m to 3.4m from the 10 to 100 year storm
2. Anecdotal reports were deemed less reliable than CEAC model predictions as a result of some inconsistencies in the anecdotal reports as well as the unusually sharp increase in the resulting setups as the return period increases. The differences could essentially be due to the quality of the topographic surface used to estimate surge levels from observation points and the accuracy of the memory of observers.
3. A CEAC storm ***surge model plus run-up prediction for the 10 to 1000yr storm was varied from 1.6 to 2.3metres above Mean Sea Level.*** This will result in inundation of the coastal areas extending to as far back as 312m, 378m, and 407m inland for the 10, 50 and 100 year return storm surges.
4. SBEACH modelling indicates that the western and central locations of the project shoreline are prone to erosion up to 38 metres from the shoreline in the 10 year return storm.

Recommendations

The following are our recommendations

1. A more detailed topographic survey and anecdotal surveys needs to be undertaken in the project area to refine storm surge elevations reported by of the residents and for inundation mapping boundaries
2. Were relevant, the necessary mitigation measures should be designed to address the erosion hazard. Additionally, building guidelines for the town, in terms of minimum floor elevations should be communicated to project proponents to ensure that future modifications and developments observe the predictions for storm surge.
3. Emergency and or evacuation plan for the community in the event of hurricanes passing close to the project area.

APPENDIX D- SEISMIC ASSESSMENT

SEISMIC ASSESSMENT OF ANNOTTO BAY USING A RAPID VISUAL SCREENING (RVS) METHODOLOGY AND THE NAKAMURA (H/V) METHOD

Dr Lyndon Brown[1]
Mrs Stephanie Grizzle[1]
Mr Rainford Grant[2]
Ms Kerri-Ann Henry[2]

1. **Earthquake Unit, University of the west Indies, Mona**
2. **Office of Disaster Preparedness & Emergency Management**

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Executive Summary

The community of Annotto Bay can be described as a coastal town. It is located in the eastern end of the parish of St. Mary. The area in general is also located at the eastern end of the island which shows the highest frequency of earthquakes in the island. The earthquake events of 1692 and 1907 reportedly generated tsunami that affected this town. The town is within twenty kilometres of the major East-west trending fault line that runs across the island, and also within the suspected epicentre of these major earthquakes. Analysis of the seismic vulnerability of the town has been assessed using two techniques, one of which us the Rapid Visual Screening (RVS) FEMA 154 which is a method of assessing the physical structure of a building by a side walk survey observing the structural and physical characteristics of the building in relation to the geophysical properties of the soil at that particular location. This guideline has been used in the USA, Asia, and Europe. The RVS method utilized a cut-off score of **2.5** for residential and commercial buildings and the assessment of the area clearly revealed that the soil is the common factor that determines the vulnerability of the community. Areas on the surrounding slope of the town that is located on harder rocks (type C) performed satisfactorily in this study and those area on low lying sections of the town on soil type D and E resulted in higher vulnerability. The second effect on the results of this study shows strong correlation to the quality of buildings and building practices. Wooden structures in general performed favourably under seismic conditions; however, the dominant construction type in the town is reinforced concrete showed varying levels of performance. The building practices impacts on the vulnerability of buildings in the town. It is clearly accepted that building irregularities, both vertical and plan, will impact the performance of structures under seismic loading. These levels of irregularity in the design of structures in the town resulted in failure of a significant number of the buildings with the RVS methodology. Critical facilities in the town were assessed with more strict guidelines (cut-off score of **3**) as these facilities are expected to respond to emergencies after an earthquake. The results of the RVS show that only 605 (60.2 %) of the residential and commercial structures passed the RVS assessment. The critical facilities fared worse in the assessment with only 33% passing the screening. Based on the RVS (FEMA 154) guideline, once structures are identified as potentially hazardous, they should be further evaluated by a professional engineer experienced in seismic design to determine if, in fact, they are seismically hazardous.

The second technique of assessment is a site effect study which uses a portable seismograph to measure the ambient ground motion; by using the horizontal to vertical ratios of ground amplification, a fundamental period/ frequency of a location is calculated. This technique described as the Nakamura method (H/V spectral ratio) is internationally used as a technique to measure seismic site effects. Both these techniques are means of measuring seismic vulnerability. The vulnerability of the town increased in relation to the soil stiffness, whereby lower stiffness results in low shear wave velocity that results in higher ground amplification based on the H/V results. Higher amplification results from weak, poorly cemented soil, and high water table, all these factors contribute to increased seismic vulnerability of an area. Results showing high ground amplification more strongly in the centre of downtown also bear guidance for future development using the concept of resonance as a guide to the most stable structures in the event of a major earthquake.

INTRODUCTION

Two approaches have been undertaken to assess the seismic vulnerability of the town of Annotto Bay. The first technique known as the Nakamura or H/V method is a technique originally proposed by Nogoshi and Igarashi (1971) and made wide-spread by Nakamura et al. (1983), and entails estimating the ratio between the Fourier amplitude spectra of the horizontal (H) to vertical (V) components of ambient noise vibrations recorded at one single station. For H/V measurements these 3 components of ground motion are required. The three- channel portable seismograph measures 3 signals: North-South, East-West and Vertical at each site.

The result of this survey identifies the fundamental frequency/period of the site. Ambient vibration recordings combined with the H/V spectral ratio technique have been proposed to help in characterising local site effects. As it is well known, occurrence of earthquake damage depends upon strength, period and duration of seismic motions and these parameters are strongly influenced by seismic response characteristics of surface ground and structures.

The H/V technique has been frequently adopted in seismic microzonation investigations. This technique is most effective in estimating the natural frequency of soft soil sites when there is a large impedance contrast with the underlying bedrock. The method is especially recommended in areas of low and moderate seismicity due to the lack of significant earthquake recordings, as compared to high seismicity areas. Site effects associated with local geological conditions constitute an important part of any seismic hazard assessment. Many examples of catastrophic consequences of earthquakes have demonstrated the importance of reliable analyses procedures and techniques in earthquake hazard assessment and in earthquake risk mitigation strategies.

The software known as Geopsy has been used to process raw field data into a H/V spectral ratio from any type of vibration signals (ambient vibrations, earthquake). A typical output from this assessment is shown in the Figure 1 below.

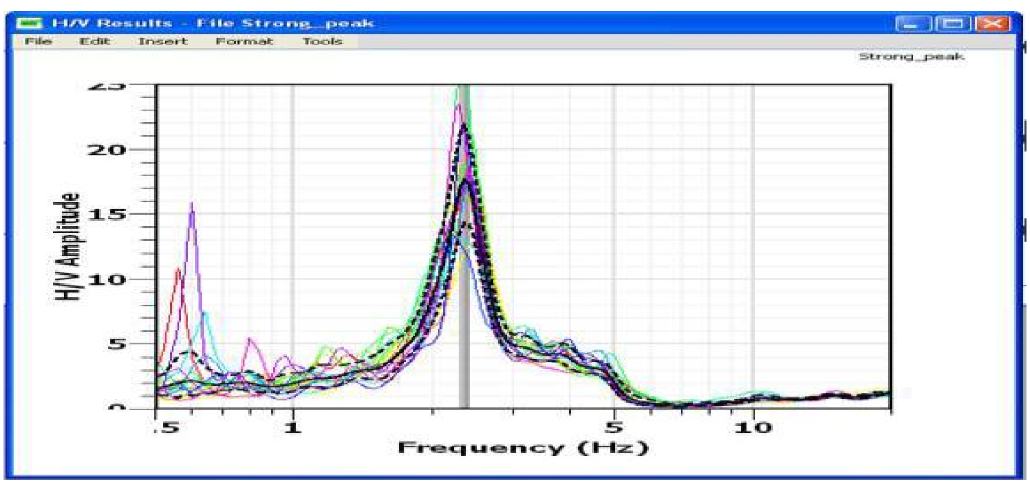


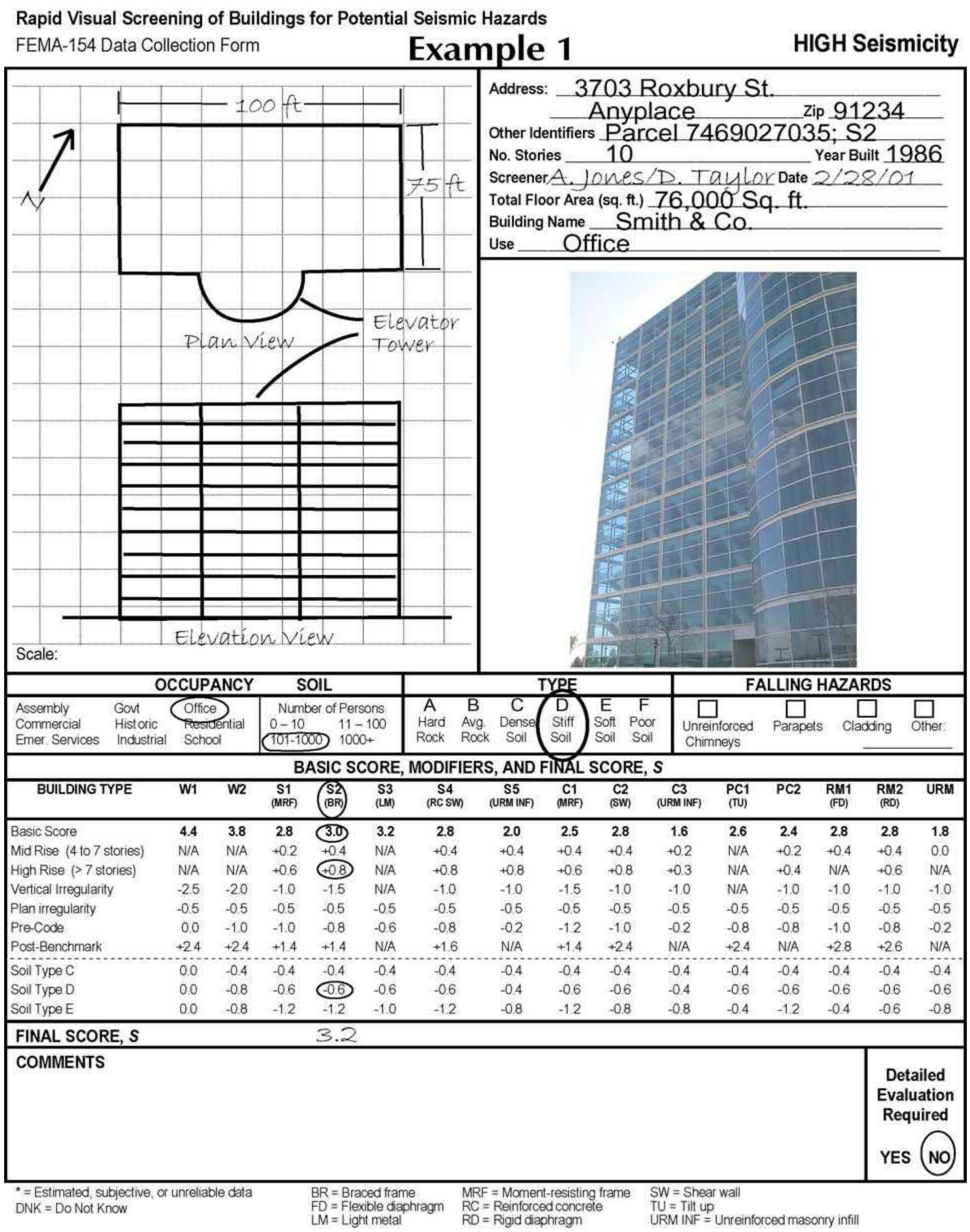
Figure 1: Typical output of the processed 3- component ambient ground motion signal to determine the fundamental frequency of a site. The period of the site is calculated by determining the inverse value of the fundamental frequency.

In figure 1 the black curve represents H/V geometrically averaged over all coloured individual H/V curves. The two dashed lines represent H/V standard deviation. The grey area represents the averaged peak frequency and its standard deviation. The frequency value is at the limit between the dark grey and light-grey areas.
The second technique is an assessment of the community by Rapid Visual Screening (RVS, 2002), a methodology developed by the United States Federal Emergency Management Agency (FEMA,). The RVS has been used by FEMA as a guideline to assess the structural integrity of buildings and this methodology has been adopted for use in India, (Sadat et. al, 2010) Turkey (Yakut 2004), Oregon (Wang and Goettel 2007) to assess the seismic vulnerability of town and cities. The RVS has been developed for use by a range of construction professionals including building officials and inspectors, and government agencies and private-sector building owners to identify, inventory, and rank buildings that are potentially seismically hazardous. The RVS uses a methodology based on a "sidewalk survey" and uses a Data Collection Form specific to the level of seismicity of the country or region, i.e. Low, Moderate or High. The person conducting the survey completes this form assigning scores based on the parameters examined or applicable to the type of building (see Table 1). This assessment is based on visual observation of the building from the exterior, and if possible, the interior. The Data Collection Form includes space for documenting building parameters, identification information, including its use and size, a photograph of the building, sketches, and documentation of pertinent data related to seismic performance, including the summation of a numeric seismic hazard score for the building based on the parameters used by the FEMA guideline to arrive at a final score of the building, see figure 2.

Although RVS is applicable to all buildings, its principal purpose is to identify (1) older buildings designed and constructed before the adoption of adequate seismic design and detailing requirements, (2) buildings on soft or poor soils, or (3) buildings having performance characteristics that negatively influence their seismic response. The intended use of the RVS procedure is to screen a population of buildings on the basis of a cut-off value after a final score is determined. The final score results separates the buildings into two categories:

- those that are expected to have acceptable seismic performance
- those that may be seismically hazardous and should be studied further

Once identified as potentially hazardous, such buildings should be further evaluated by a professional engineer experienced in seismic design to determine if, in fact, they are seismically hazardous.



Rapid Visual Screening of Buildings for Potential Seismic Hazards
FEMA-154 Data Collection Form

Example 1

HIGH Seismicity

Address: 3703 Roxbury St.
Anyplace Zip 91234
Other Identifiers Parcel 7469027035; S2
No. Stories 10 Year Built 1986
Screener A. Jones/D. Taylor Date 2/28/01
Total Floor Area (sq. ft.) 76,000 Sq. ft.
Building Name Smith & Co.
Use Office

OCCUPANCY — Assembly, Govt, Office, Commercial, Historic, Residential, Emer. Services, Industrial, School

SOIL — Number of Persons: 0 – 10, 11 – 100, 101-1000, 1000+

TYPE — A Hard Rock; B Avg. Rock; C Dense Soil; D Stiff Soil; E Soft Soil; F Poor Soil

FALLING HAZARDS — Unreinforced Chimneys; Parapets; Cladding; Other:

BASIC SCORE, MODIFIERS, AND FINAL SCORE, S

BUILDING TYPE	W1	W2	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RC SW)	S5 (URM INF)	C1 (MRF)	C2 (SW)	C3 (URM INF)	PC1 (TU)	PC2	RM1 (FD)	RM2 (RD)	URM
Basic Score	4.4	3.8	2.8	3.0	3.2	2.8	2.0	2.5	2.8	1.6	2.6	2.4	2.8	2.8	1.8
Mid Rise (4 to 7 stories)	N/A	N/A	+0.2	+0.4	N/A	+0.4	+0.4	+0.4	+0.4	+0.2	N/A	+0.2	+0.4	+0.4	0.0
High Rise (> 7 stories)	N/A	N/A	+0.6	+0.8	N/A	+0.8	+0.8	+0.6	+0.8	+0.3	N/A	+0.4	N/A	+0.6	N/A
Vertical Irregularity	-2.5	-2.0	-1.0	-1.5	N/A	-1.0	-1.0	-1.5	-1.0	-1.0	N/A	-1.0	-1.0	-1.0	-1.0
Plan irregularity	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
Pre-Code	0.0	-1.0	-1.0	-0.8	-0.6	-0.8	-0.2	-1.2	-1.0	-0.2	-0.8	-0.8	-1.0	-0.8	-0.2
Post-Benchmark	+2.4	+2.4	+1.4	+1.4	N/A	+1.6	N/A	+1.4	+2.4	N/A	+2.4	N/A	+2.8	+2.6	N/A
Soil Type C	0.0	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4
Soil Type D	0.0	-0.8	-0.6	-0.6	-0.6	-0.6	-0.4	-0.6	-0.6	-0.4	-0.6	-0.6	-0.6	-0.6	-0.6
Soil Type E	0.0	-0.8	-1.2	-1.2	-1.0	-1.2	-0.8	-1.2	-0.8	-0.8	-0.4	-1.2	-0.4	-0.6	-0.8
FINAL SCORE, S				3.2											

COMMENTS

Detailed Evaluation Required YES NO

* = Estimated, subjective, or unreliable data
DNK = Do Not Know
BR = Braced frame
FD = Flexible diaphragm
LM = Light metal
MRF = Moment-resisting frame
RC = Reinforced concrete
RD = Rigid diaphragm
SW = Shear wall
TU = Tilt up
URM INF = Unreinforced masonry infill

Figure 2: Completed assessment of a building using the RVS in a high seismicity zone.)

Parameters Considered In RVS

The parameters used in screening buildings to determine the total numerical score of a building includes the seismic hazard intensity, building type, height of the building, soil type in the foundation, plan and vertical irregularity of the building, conformity to the seismic building code in the design; see Table 2 for discussion of the properties of these modifiers).

Each Hazard Intensity Form (Low, Moderate, or High) has separate scoring values for each building type and each score vary for each modifier for each building type. The building type is assigned an initial basic score which is in fact related to its lateral load resisting structural system and earthquake performance and then additional modifying scores (only those specific to the building and soil type) are added or subtracted from the basic score to arrive at a final score for each building in the assessment. Table 3 shows the different soil types (with explanation of the geophysical characteristics) that defines the soil modifier in the different sections of the community.

Table 1: FEMA Classification Building Type, Fifteen Building Types Considered by the RVS Procedure

Building Code	Building Description	Building Code	Building Description
W1	Light wood-frame residential and commercial buildings smaller than or equal to 5,000 square feet	C2	Concrete shear-wall buildings
W2	Light wood-frame buildings larger than 5,000 square feet	C3	Concrete frame buildings with unreinforced masonry infill walls
S1	Steel moment-resisting frame buildings	PC1	Tilt-up buildings
S2	Braced steel frame buildings	PC2	Precast concrete frame buildings
S3	Light metal buildings	RM1	Reinforced masonry buildings with flexible floor and roof diaphragms
S4	Steel frame buildings with cast-in-place concrete shear walls	RM2	Reinforced masonry buildings with rigid floor and roof diaphragms
S5	Steel frame buildings with unreinforced masonry infill walls	URM	Unreinforced masonry bearing-wall buildings (Also made to include ***Wattle and Daub*** structures – building technique which utilizes a woven lattice of wood strips daubed with wet soil such as clay and straw.)
C1	Concrete moment-resisting frame buildings		

After a complete assessment is done of a building, a final score is obtained which determines the expected seismic performance of that building. The cut-off score and final score of the structure indicates if the building is seismically safe or unsafe. If unsafe detailed engineering assessment is required. The cut-off score used in this study for non-critical facilities was 2.5 which is a little higher than FEMA's typical score of 2.0. A greater score was chosen due to the fact that the study area is located in the section of the island that has highest frequency of seismic activities. Mathematically, a final score of 2.0 means an estimated 1% chance of collapse at the defined level of ground shaking in the area of the country where the building is located.

The scores are logarithmically related to the likelihood of complete structural damage (and collapse), but suffice it to say that a number above 2.5 means the building probably represents a low collapse risk in an extreme earthquake, and a number below 2.5 means the building is of enough concern to warrant a detailed seismic evaluation by a qualified structural engineer. One of the more difficult steps in the RVS procedure is determining the cut-off score, since it poses the question involving the cost of safety versus the benefits. In general, buildings which fall in the category of emergency services are normally given a cut-off score of 3 which indicates that the buildings with a score of 3 or more would have a 1 in 10^3 chance of receiving severe damage in the event of major earthquake. There are several factors to consider when selecting a cut-off score for a region. The present state of the country's economy is one factor that is considered when selecting a cut-off score. The economic stability of the country becomes relevant in the decision process because the higher the cut-off score the more likely for building's final score to fall below the threshold value. Structures which do not meet the cut-off score would therefore require a detailed evaluation to be done, which can be very costly as professional personnel with specialized equipment would be employed to determine the potential of seismic hazards (FEMA -154).

Table 2: Description of applicable modifiers used in scoring the performance of each building

Modifier	**Modifiers Description**
Mid-Rise	4-7 Storeys
High-Rise	8 or more storeys
Vertical irregularity	Hillside buildings, soft storeys, irregular shape in elevation
Plan irregularity	Buildings with re-entrant corners, buildings with good lateral-load resistance in one direction but not in the other; and buildings with major stiffness eccentricities in the lateral force- resisting system, L shaped, T-Shaped, U-shaped, large openings, Weak Link Between Larger Building Plan Areas
Pre-Code	buildings in high and moderate seismicity regions and is applicable if the building being screened was designed and constructed prior to the initial adoption and enforcement of seismic codes applicable for that building type
Post-Benchmark	Building designed and constructed after significantly improved seismic codes applicable for that building type (e.g., concrete moment frame, C1)
Soil Type	Score Modifiers are provided for Soil Type C, Type D, and Type E. The appropriate modifier should be circled if one of these soil types exists at the site

Table 3: Soil Type Definitions and Related Parameters

Soil Type Definitions	**Related Parameters**
Type A (hard rock)	Measured shear wave velocity (*vs)* **>** 5000 ft/sec.
Type B (rock)	*vs* between 2500 and 5000 ft/sec.
Type C (soft rock and very dense soil)	*vs* between 1200 and 2500 ft/sec, or standard blow count(*N)* **>** 50, or undrained shear strength (*su)* **>** 2000 psf.
Type D (stiff soil)	*vs* between 600 and 1200 ft/sec, or standard blow count (*N)* between 15 and 50, or undrained shear strength (*su)* between 1000 and 2000 psf.
Type E (soft soil)	More than 100 feet of soft soil with plasticity index (PI) > 20, water content (w) > 40%, and *su* **<** 500 psf; or a soil with *vs* ≤ 600 ft/sec.

Type F (poor soil)	Soils requiring site-specific evaluations:

History of Earthquake in Jamaica

The north-eastern section of the island has the highest frequency of earthquake activity. Although the precise epicentre of the major earthquakes of 1692 and 1907 are not known, based on intensity reports it is theorized that a likely location for the epicentres for these events would be in the north-eastern section of the island. There were also reports of tsunami in Annotto Bay after the 1907 which also adds credence to the theory that the epicentres of this event was within the north-eastern section of the island.

Based on the EQU focal depth solutions it is clear that the hypocentre of earthquake events in this area are typically very shallow (~ 15 km) with typical fault offset in a left lateral motion. Focal mechanism solutions of recent earthquakes indicate a left lateral motion to be responsible for most fault dynamics.

Relatively high frequencies of earthquake occur in the JSN sub-area (see figure 3); direct epicentres of event are rare in the town of Annotto Bay. However, even though there are no clear events in this location, the source area of most active events in the island is within 20 kilometres of this town. Thus very shallow earthquake and short distance of active faults coupled with poor construction practices, spells a high probability of serious earthquake damage for the study area.

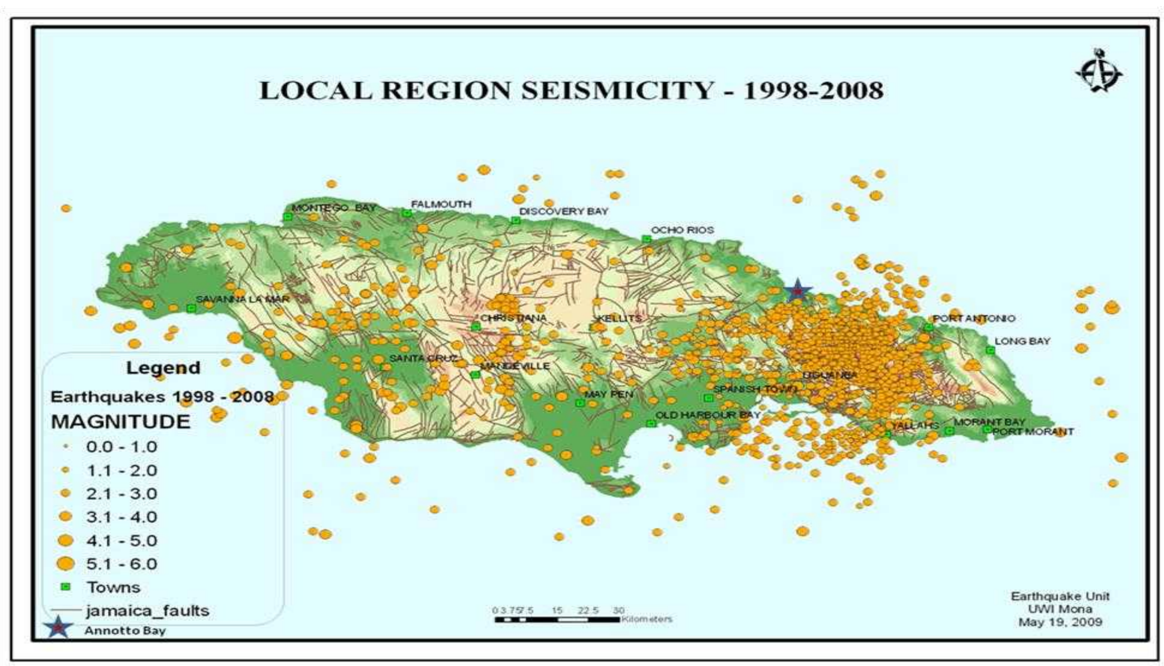


Figure 3: Epicentres of earthquakes in Jamaica during 1998-2008, highest frequency of events occurs in the north-eastern section of the island in close proximity to study area.

Geology of Annotto Bay

The surficial geological units in the study area are alluvial deposits overlying White Limestone of the Gibraltar- Bonny Gate Formation and the Richmond Formation.

The alluvial deposits occur along the coast and lower reaches of the major rivers that terminate along the coastal flats in the town of Annotto Bay. The deposits range from carbonaceous to silica rich sands with abundant shell fragments that vary in thickness and reaches up to 70 metres in the western section of the town in the Aqualtavale area.

The Richmond Formation outcrops towards the western section of the town. It is composed of a series of well bedded grey to brown–weathering alternating calcareous sandstones, siltstones and mudstones with occasional thin beds of limestone and massive conglomerates. The Gibraltar- Bonny Gate Formation outcrops towards the eastern end of the town and can be described as a series of evenly bedded chalky limestone with occasional bioclastic layers.

Faulting is the dominant structural feature in the area with the longest fault lines being two unnamed major faults showing a north-south trend which appears at the eastern limit of the Wagwater Formation (see Figures 4 & 7). The Vere-Annotto Bay fault is also a major fault in the study area with a SW-NE trend; however, there are no surficial expressions of this fault line. Furthermore, several faults with a general east-west trend are also dominant in the area. Minor surface expression of the fault is shown in the area and the boundary of the more easterly Gibraltor Formation and Richmond Formation is marked by an east west trend minor fault that borders the slopes surrounding the town from the northerly coastal flats.

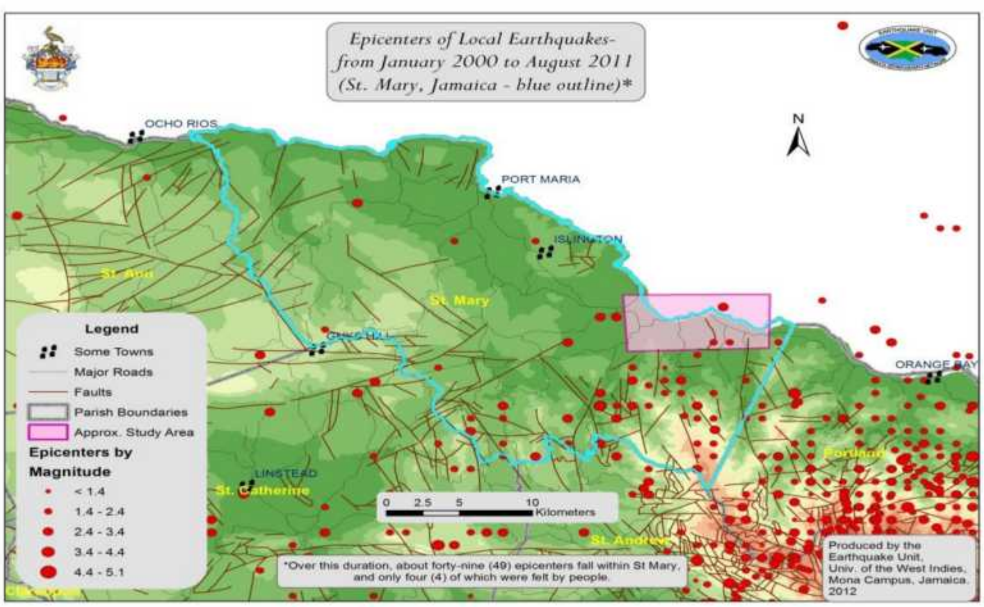


Figure 4: Major faults and epicentres of earthquake events (2000-2011) are shown in relation to the study area.

Result of H/V Field Data

There are 14 points where measurements were done in Annotto Bay, see Figure 5. The readings were taken at approximately 500 metres between each point.

Portable seismograph (Guralp 40 T) instrument was used to collect ambient ground motion. For each of these sites care was taken to ensure that the location had minimal noise (anthropogenic or natural). The Instrument was left to stand at each point on average 30 minutes so that good quality data could be collected over this period. The processed wave form for each site is shown in Figure 6, with maximum amplitude in relation to the fundamental frequency of each site. Having calculated the fundamental frequency, the period (inverse of frequency) of each site was also calculated, see table 4.

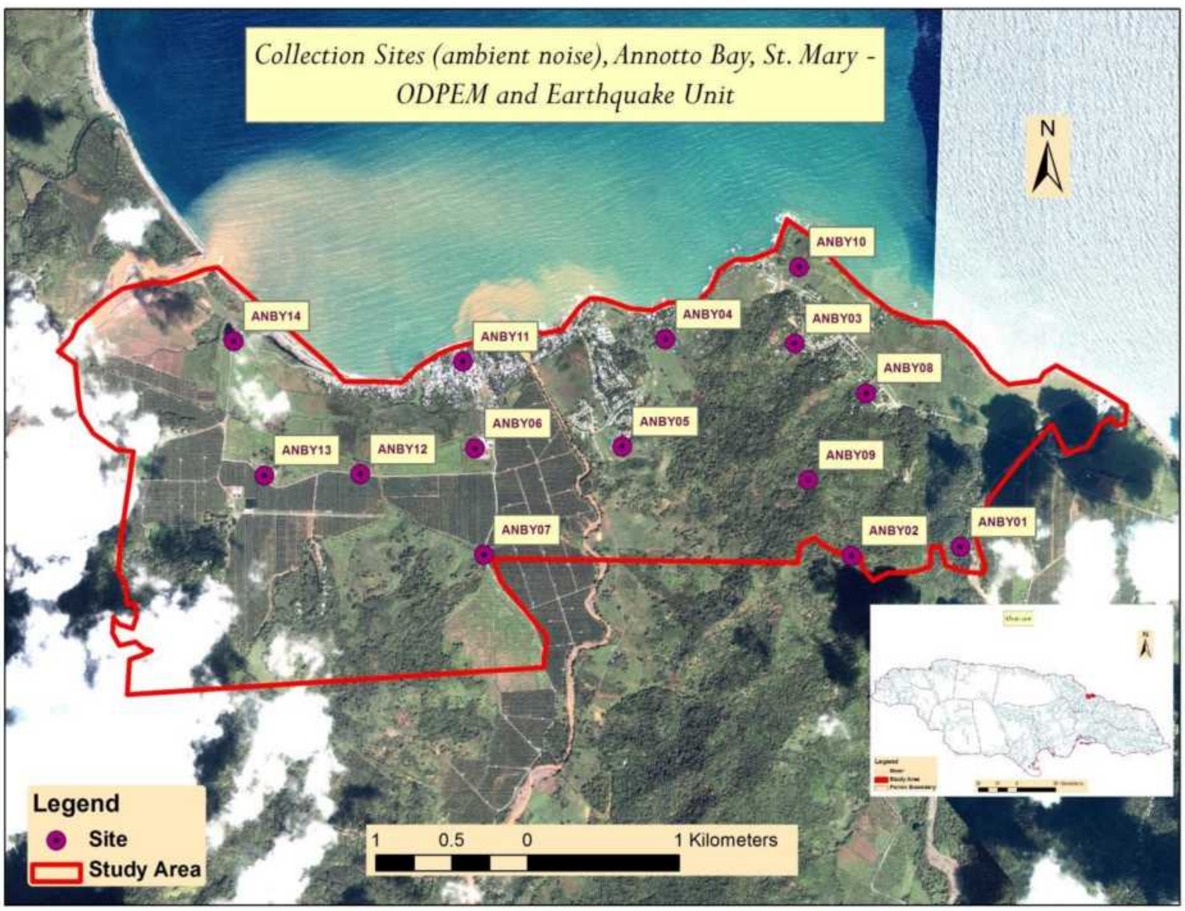
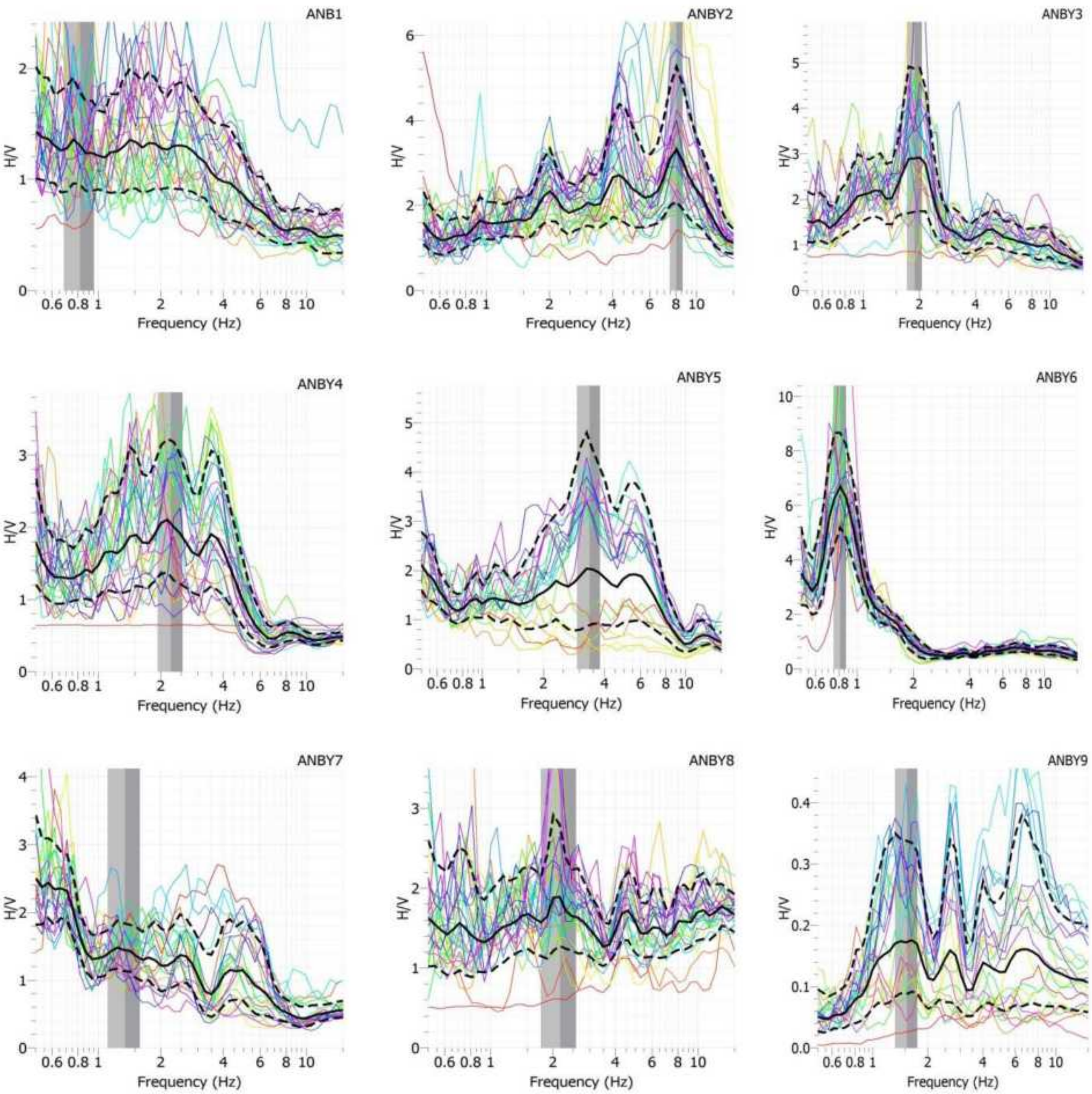


Figure 5: Location of areas where a site effect study was done in Annotto Bay.


ANB1
ANBY2
ANBY3
ANBY4
ANBY5
ANBY6
ANBY7
ANBY8
ANBY9
H/V
Frequency (Hz)

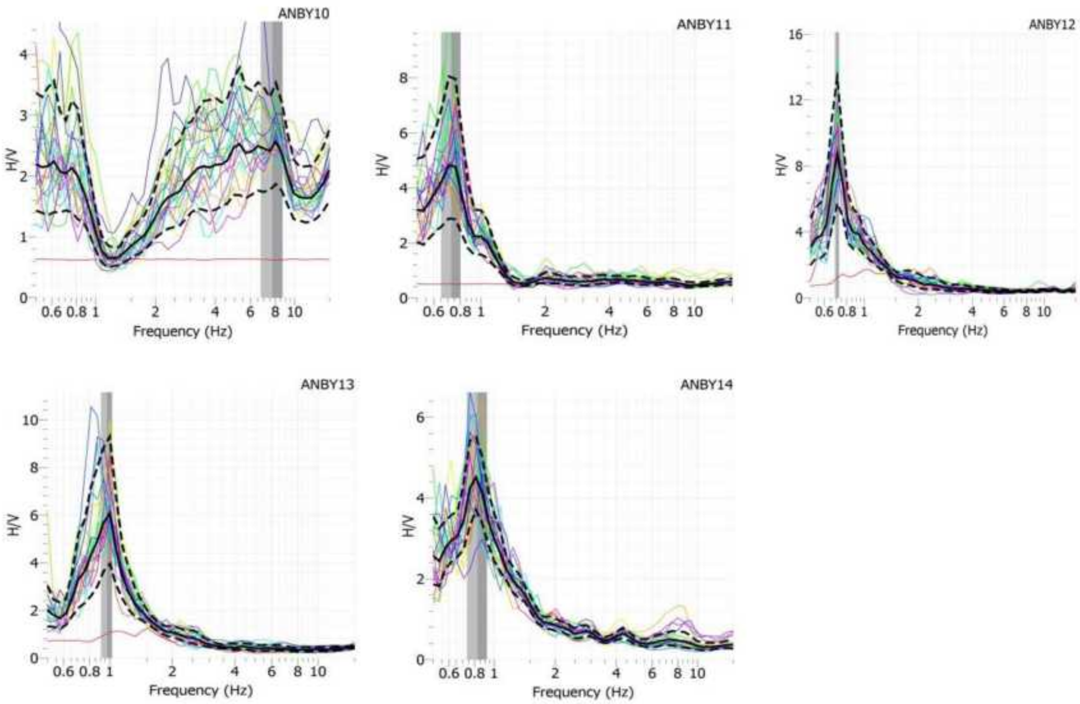


Figure 6: H/V spectral ratio of each site within the study area.

Table 4: Period of each site determined from the fundamental frequency of each H/V spectral ratio.

Site	**Fundamental Frequency (Hz)**	**Period (s)**
ANBY01	0.7	1.4
ANBY02	8	0.1
ANBY03	3	0.3
ANBY04	2	0.5
ANBY05	3.5	0.3
ANBY06	0.7	1.4
ANBY07	1.5	0.7
ANBY08	2	0.5
ANBY09	1.5	0.7
ANBY10	8	0.1
ANBY11	0.7	1.4
ANBY12	0.5	2
ANBY13	1	0.5
ANBY14	0.8	1.3

A complete map based on the iso-period of the site is shown in Figure 7.

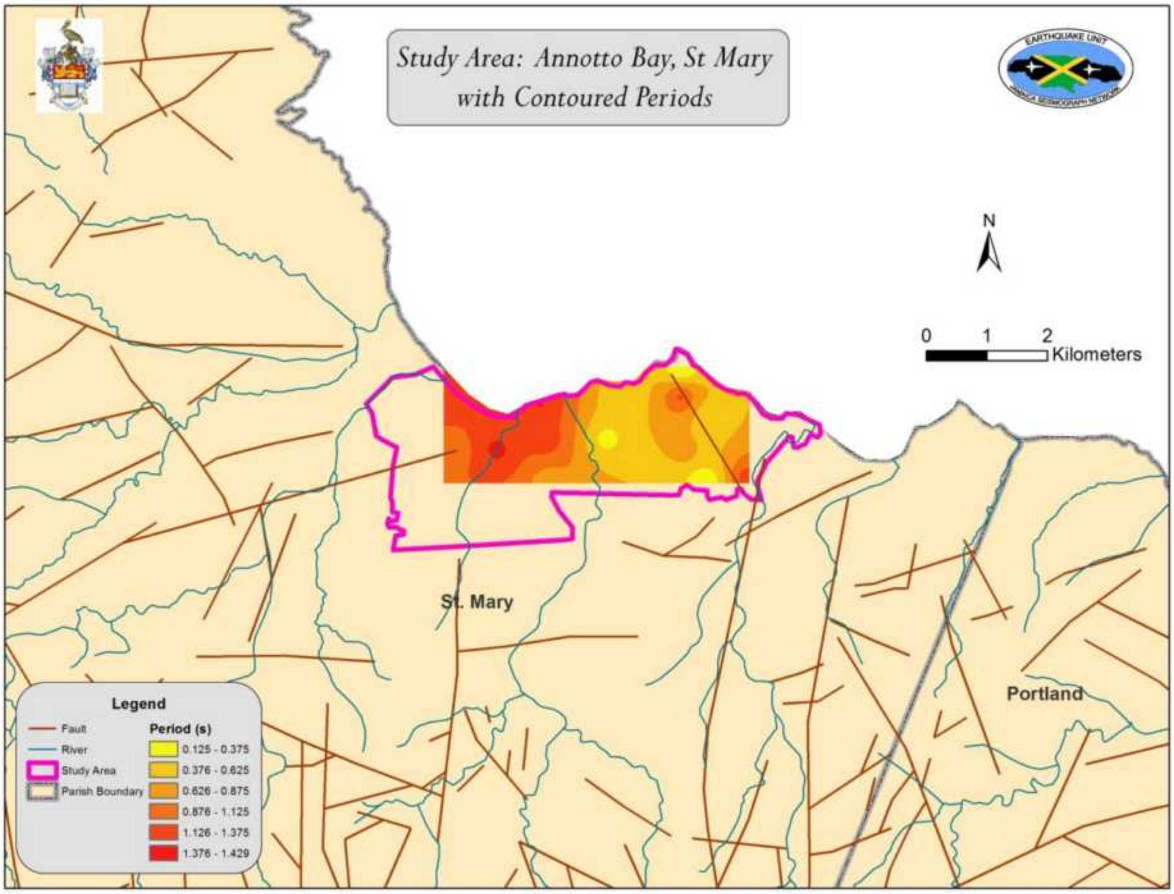


Figure 7: Map of iso-period based on H/V methodology of the study area.

Summary of the H/V Assessment

It has long been known that the effects of local geology on ground shaking represent an important factor in earthquake engineering. In particular, soft sedimentary cover could strongly amplify the seismic motion. The frequency band affected by such effects depends on the thickness and on the velocity of the sedimentary layers. When amplifications occur at frequencies close to the fundamental frequency of vibration of the buildings greater damages can be expected.

During an earthquake buildings oscillate, but not all buildings respond to an earthquake equally. If the frequency of oscillation of the ground is close to the natural frequency of the building, resonance (high amplitude continued oscillation) may cause severe damage.

In the analysis of the H/V data attention was paid to past research where most examples reported in the literature indicate clear peaked H/V curve for soft soils and almost flat curves for rock sites. When the H/V peak is clear, then the site under study presents a large velocity contrast at some depth, and is very likely to amplify the ground motion.

Based on the H/V curves spectral ratios data in Annotto Bay there are sections in the town (see figure 5) that shows clear single frequency patterns with high amplification (Sites ANBY6, ANBY9,ANBY11, ANBY12, ANBY13, ANBY14) indicating characteristics of thick soil layer. These areas are expected to show high amplification in a major earthquake. Based on the period pattern in these communities there should no serious issues with resonance as the ratio between the resonance effect (0.1 sec/single storey) does not exist. Most buildings in the community are within 1-2 stories and the period pattern falls above these ratios so the issue of resonance is not a critical factor.

For future development, attention must be paid to the height of structures and the fundamental period of the different areas of the community as stability of buildings bear clear correlation with resonance.

RVS: STRUCTURAL PERFORMANCE WITH IN STUDY AREA

Classification of Buildings in Annotto Bay

The study area consists of nine building types namely: Reinforced concrete, Wood, W/Concrete, URM (Brick), Nog, Nog & Concrete, Wood/Nog/Concrete, Wattle and Daub and metal containers modified to serve mainly as small commercial buildings, see figure 8.

A total of 1498 buildings were identified in the community. The three main building types include; Reinforced Concrete (970), Wood (443) and W/Concrete (69). These structures represented 64.8%, 29.6% and 4.6% percentage concentration respectively, while the remaining six building types had less than 0.1% concentration except for containers that had a concentration of 0.6%. See Figures 8-10 of building distribution in study area.

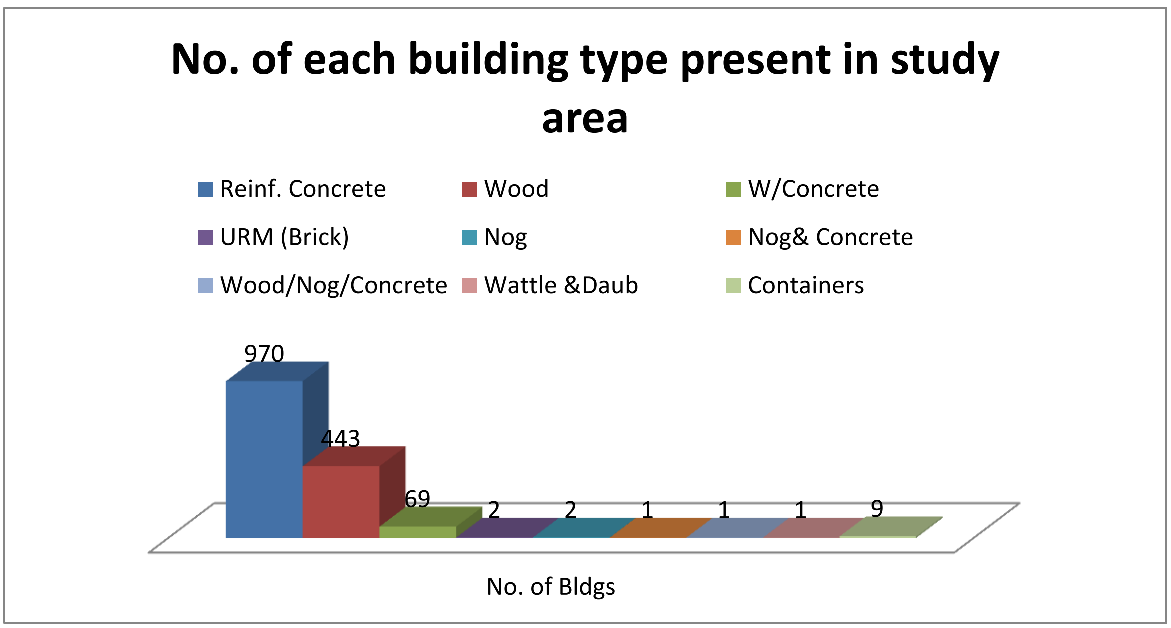
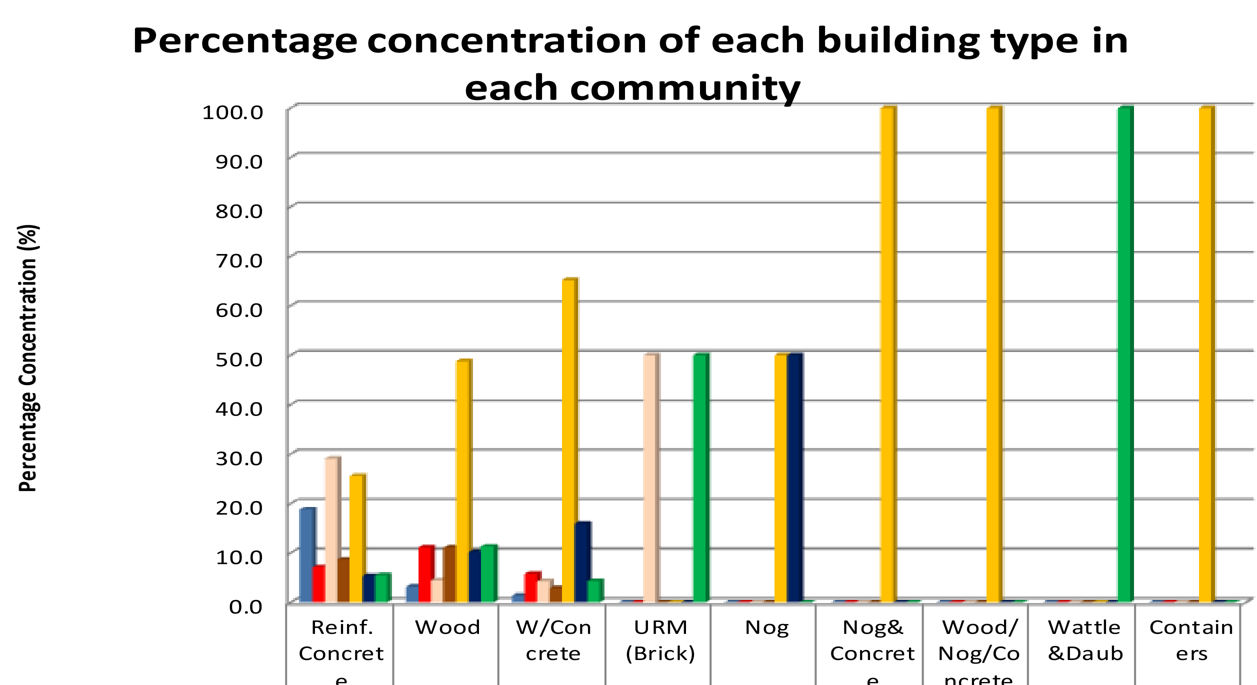


Figure 8: Column chart illustrating the number of each building type in the study area.



	Reinf. Concrete	Wood	W/Concrete	URM (Brick)	Nog	Nog& Concrete	Wood/ Nog/Concrete	Wattle &Daub	Containers
A3A - A3T	18.8	3.2	1.4	0	0	0	0	0	0
(A3F - A3H)	7.1	11.1	5.8	0	0	0	0	0	0
(A3H - A3J)	29.1	4.5	4.3	50	0	0	0	0	0
(A3J - Pencar River)	8.7	11.1	2.9	0	0	0	0	0	0
Pencar River to Annotto River	25.6	48.8	65.2	0	50	100	100	0	100
Annotto River to Fire Station End	5.4	10.2	15.9	0	50	0	0	0	0
Grays Inn	5.5	11.3	4.3	50	0	0	0	100	0

Figure 9: Column chart illustrating the percentage concentration of each building type in each community.

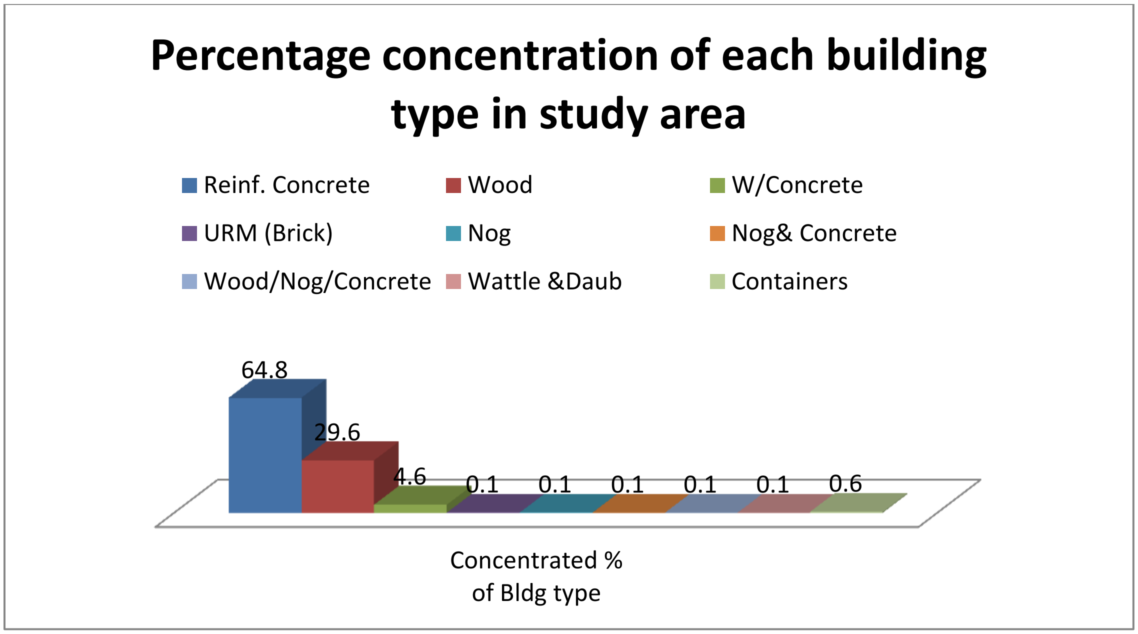


Figure 10: Column chart showing the percentage concentration of each building type in the study area

RVS DATA ANALYSIS

For the purpose of this assessment the town of Annotto Bay was divided into a number of area/ section as per geographic boundaries. Boundaries include community with distinct borders or development within a set area, demarcation by development such as a housing-scheme or demarcation by geographic features such as rivers. Based on this scheme the town was divided into seven (7) sections as follows:

1. **A3A-A3T:** *Iterboreale Community*
   - **Boundary:** [N=18.26931, W= 76.74412
   - N=18. 27144, W=76.74406
   - N=18.27255,W=76.74783
   - N=18.27104,76.77284]
2. **A3F-A3H:** *Demarcated by Iter Boreale to the west and the Hospital Road to the east*
   - **Boundary:** [N=18.27717,W=76.75246
   - N=18.27472, W=76.75774
   - N=18.27344, W=7675836]

3. ***A3H - A3J:*** *Encapsulates the Hospital and Gibraltar Housing Scheme*

    - **Boundary: [**N=18.27330, W=76.76338
    - N=18.26764, W=76.75789
    - N=18.27075, W=76.76074
    - N=18.26566, W=76.76018
    - N=18.26717, W=76.76274**]**

4. **A3J-Pencar River:** *Gibraltar Housing Scheme Road to Pencar River*

    - **Boundary;** [N=18.27015, W=76.76554]

5. **Pencar River - Annotto River:** *community located between Pencar River and Annotto River including buildings on both sides of arterial road.*

    - **Boundary [**N=18.20715, W=76.76982
    - N=18.27104, W=76.77284
    - N=18.27194, W=76.77203
    - N=18.26601, W=76.76974**]**

6. **Annotto River - Fire Station End:** *All structures located on both sides of the arterial road from the Annotto River to the Fire Station.*

    - **Boundary: [**N=18.26990, W=76.77496
    - N=18.26978, W=76.77522
    - N=18.27050, W=76.77946**]**

7. **Grays Inn:** this include structures west of Grays Inn Football field and to the western end of Study area.
    - **Boundary:** [N=18.25736, W=76.77946
    - N=18.26053, W=76.78020**]**

The presentation below outlines the performance of the nine types of structures within the town. Each variation of the specific building class is scored based on the applicable RVS modifiers. A post benchmark modifier was applied to those buildings that were built in the post 1983 when engineers applied regional building codes. However, in cases where this criterion was met but buildings were of poor quality (indicated by poor construction method or structural defects that would imply that adequate engineering consideration was absent). Hence, the final score produced by these structures would best reflect the average performance expected for these types of structures.

Reinforced masonry buildings with rigid floor and roof diaphragms (RM1)

Based on the RVS guidelines, the RM1 structures are given a basic score of 3.6 and the final score is calculated based on the applicable modifiers. These RM1 structures exist on soil types C-E, and the corresponding final scores were calculated for each of the structures found in each area based on soil type. See tables 5-7 for the results of structural performance of RM1 buildings based on varied combinations of existing modifiers and soil type.

Table 5: Qualitative assessment of **RM1** buildings based on applicable modifiers and resulting final scores on **Soil type C**.

Base score of Building / Modifier Scores	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
Vertical irregularity	-2.0	-2.0	-	-	-2.0	-2.0	-	-	-
Plan irregularity	-0.5	-0.5	-	-0.5	-0.5	-	-	-	-
Soil type C	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8
Post benchmark	2.0	-	2.0	2.0	-	2.0	-	-	-
Pre-code	-	-	-	-	-0.4	-	-0.4	-	-0.4
Final Score	**2.3**	**0.3**	**4.8**	**4.3**	**-0.1**	**2.8**	**2.4**	**2.8**	**2.4**

Table 6: Qualitative assessment of **RM1** buildings based on applicable modifiers and resulting final scores on **Soil type D.**

Base score of Building / Modifier Scores	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
Vertical irregularity	-2.0	-2.0	-	-	-2.0	-	-	-
Plan irregularity	-0.5	-0.5	-	-0.5	-0.5	-	-	-
Soil type D	-1.2	-1.2	-1.2	-1.2	-1.2	-1.2	-1.2	-1.2
Post benchmark	2.0	-	2.0	2.0	-	-	-	-
Pre-code	-	-	-		-0.4	-0.4	-	-0.4
Final Score	**1.9**	**-0.1**	**4.4**	**3.9**	**-0.5**	**2.0**	**2.4**	**2.0**

Table 7: Qualitative assessment of **RM1** buildings based on applicable modifiers and resulting final scores on **Soil type E**.

Base score of Buildings / Modifier Scores	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
Vertical irregularity	-2.0	-2.0	-	-	-2.0	-	-	-
Plan irregularity	-0.5	-0.5	-	-0.5	-0.5	-	-	-
Soil type E	**-1.6**	**-1.6**	**-1.6**	**-1.6**	**-1.6**	**-1.6**	**-1.6**	**-1.6**
Post benchmark	2.0	-	2.0	2.0	-		-	
Pre-code	-	-			-0.4	-0.4	-	-0.4
Final Score	**1.5**	**-0.5**	**4.0**	**3.6**	**-0.9**	**1.6**	**2.0**	**1.6**

Unreinforced Masonry bearing-wall buildings (URM)

Brick, Nog and Wattle and Daub structures are all considered as unreinforced masonry (URM) and as such the final score is calculated using the URM category. It should also be noted that post benchmark doesn't apply to URM structures when using the moderate seismicity form. The basic score applied to URM structures is a score of 3.4, and as illustrated in the previous description of RM1 structures. Applicable modifiers are added to the score based on the variation of the building designs and soil types found in the specific area. Tables 8-10, illustrate the final scores produced by these structures for combination of modifiers that were applied for structures in the area.

Table 8: Qualitative assessment of **URM** buildings based on applicable modifiers and resulting final scores on **Soil type C**.

Base score of Building / Modifier Scores	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
Vertical irregularity	-1.5	-1.5	-	-	-1.5	-	-	-
Plan irregularity	-0.5	-0.5	-	-0.5	-0.5	-	-	-
Soil type C	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4
Post benchmark	2.0	-	2.0	2.0	-	-	-	-
Pre-code	-	-	-	-	-0.4	-0.4	-	-0.4
Final Score	**3.0**	**1.0**	**5.0**	**4.5**	**0.6**	**2.6**	**3.0**	**2.6**

Table 9: Qualitative assessment **URM** building based on applicable modifiers and resulting final scores on **Soil type D**

Base score of Building / Modifier Scores	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
Vertical irregularity	-1.5	-1.5	-	-	-1.5	-	-	-
Plan irregularity	-0.5	-0.5	-	-0.5	-0.5	-	-	-
Soil type D	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8
Post benchmark	2.0	-	2.0	2.0	-	-	-	-
Pre-code	-	-	-	-	-0.4	-0.4	-	-0.4
Final Score	**2.6**	**0.6**	**4.8**	**4.1**	**0.2**	**2.2**	**2.6**	**2.2**

Table 10: Qualitative assessment **URM** building based on applicable modifiers and resulting final scores on **Soil type E**.

Base score of Building / Modifiers Score	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
Vertical irregularity	-1.5	-1.5	-	-	-1.5	-	-	-
Plan irregularity	-0.5	-0.5	-	-0.5	-0.5	-	-	-
Soil type E	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6
Post benchmark	2.0	-	2.0	2.0	-	-	-	-
Pre-code	-	-	-	--	-0.4	-0.4	-	-0.4
Final Score	**1.8**	**-0.2**	**3.8**	**3.3**	**-0.6**	**1.4**	**1.8**	**1.4**

Steel moment - resisting frame buildings (S1)

S1 structures also included the metal containers that were modified primarily for use as small commercial buildings. Based on the RVS guideline S1 structures are given a basic score of 3.6. These structures were only found on soil type E and as such the appropriate modifiers were applied based on this soil type. Table 11 shows the resulting final score for these structures based on varied combination of modifiers that existed in the area.

Table 11: Qualitative assessment of **S1** buildings based on applicable modifiers and resulting final scores on **soil type E.**

Base score of Building / Modifiers Score	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
Vertical irregularity	-1.5	-1.5	-	-	-1.5	-	-	-
Plan irregularity	-0.5	-0.5	-	-0.5	-0.5	-	-	-
Soil type E	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8
Post benchmark	2.0	-	2.0	2.0	-	-	-	-
Pre-code	-	-	-	-	-0.4	-0.4	-	-0.4
Final Score	**2.8**	**0.8**	**4.8**	**4.3**	**0.4**	**2.4**	**2.8**	**2.4**

Concrete moment-resisting frame buildings (C1)

C1 structures are given a basic score of 3.0 and similarly the applicable modifiers are added to the basic score as dictated by design and soil type. These structures were only found on soil type E and the resultant final scores for the varied combination of modifiers are illustrated in Table 12.

Table 12: Qualitative assessment of **C1** buildings based on applicable modifiers and resulting final scores on **Soil type E.**

Base score of Building / Modifiers Score	3.0	3.0	3.0	3.0	3.0
Vertical irregularity	-2.0	-2.0	-	-	-
Plan irregularity	-0.5	-	-0.5	-	-
Soil type E	-1.6	-1.6	-1.6	-1.6	-1.6
Post benchmark	-	-	-	-	
Pre-code	-	-	-	-	-0.4
Final Score	**-1.1**	**- 0.6**	**0.9**	**1.4**	**2.2**

Light wood-frame residential and commercial buildings ≤ 5,000 square feet (W1)

W1 structures are generally very good seismic performers and has a basic score of 5.2; however, a large number of the structures in the study area were not properly constructed (make-shift plywood) and as such adjustments were made to factor in the likely reduced seismic performance. Usually a pre-code would normally be applied to adjust this shortcoming; however, FEMA does not provide a pre-code modifier for W1 structures. Therefore, the basic score for W2 was used to calculate the final score of W1 structures that fits the aforementioned scenario. W1 structures were found on all three soil types (C, D and E). As W1 structures are generally very good seismic performers only the combined modifiers scenarios existing on soil type E (worst case scenario) are illustrated in Table 13 below.

Table 13: Qualitative assessment of **W1** building based on applicable modifiers and resulting final scores of buildings on soil type **E.**

Base score of Building / Modifiers Score	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
Vertical irregularity	-3.5	-3.5	-	-3.5	-	-3.5	-	-	-
Plan irregularity	-0.5	-0.5	-	-	-0.5	-0.5	-	-	-
Soil type E	-1.2	-1.2	-1.2	-1.2	-1.2	-1.2	-1.2	-1.2	-1.2
Post benchmark	1.6	-	1.6	1.6	1.6	-	-	-	-
Pre-code	-	-	-	-	-	-0.4	-0.4	-	-0.4
Final Score	**1.2**	**-0.4**	**5.2**	**1.7**	**4.7**	**-0.8**	**3.2**	**3.6**	**3.2**

RVS PERFORMANCE BY COMMUNITIES

The community of Iterboreale

The underlying soil type in the community of Iterboreale includes type C and D, and housed only three building types namely RM1, W1 and W/Concrete. Reinforced concrete structures in this area reflected moderate to good seismic performance producing final performance scores between 3.9 and 4.3.. This is a relatively young community (most building in this area is less than 15 years) these structures were recently and properly constructed, thus the post code modifier was applied. However, in circumstances where vertical irregularities were present they reflected seismic vulnerability with reduced performance scores ranging from 1.9 and 2.3.. On the other hand the W/Concrete structure existed only on soil type D and proved to be seismically vulnerable as it produced a final score of 2.0

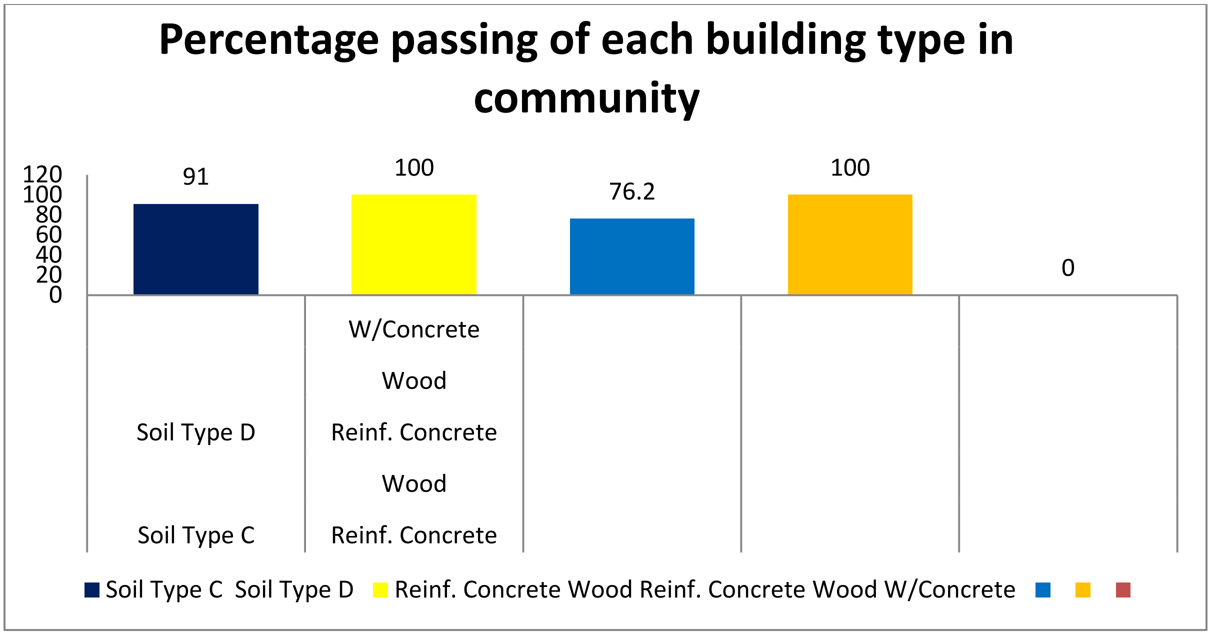


Figure 11*:* Column chart illustrating percentage passing of each building type when modifiers are applied.

Additionally, wood structures found in this area reflected very sound performance in both soil type C and D producing final scores of at least 4.6 and 3.7 respectively. (See Table 14)

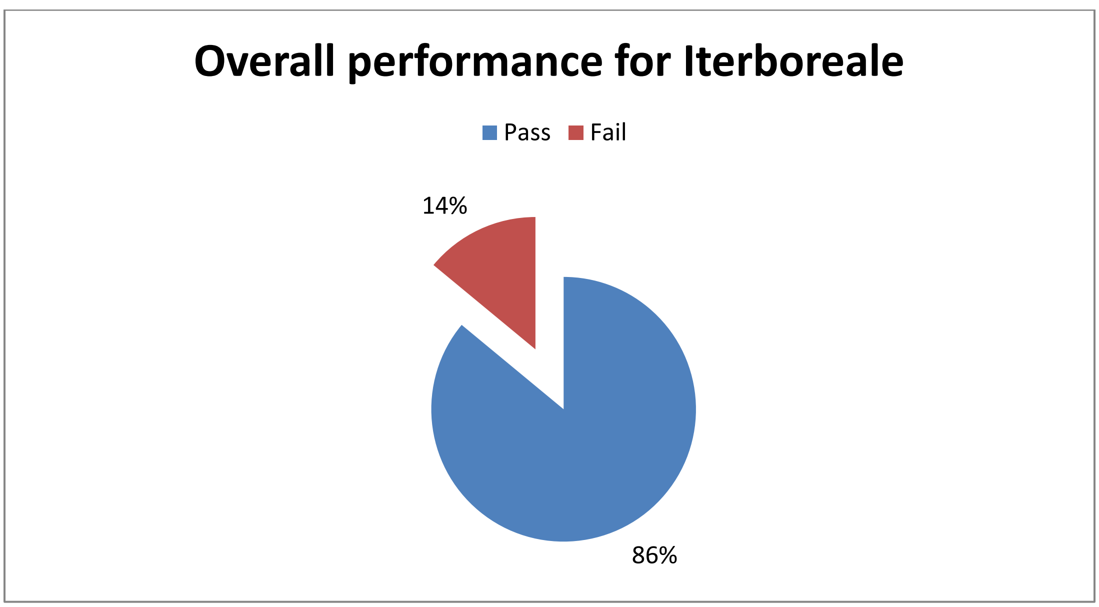


Figure 12: Pie chart illustrating overall performance of structures in the community of Iterboreale

As most of the structures found in this area were newly constructed reinforced concrete structures (92% concentration) and most of these were located on Soil type C, eighty six percent (86%) of the overall 197 structures passed (no detailed assessment is necessary) with the lowest final score being 3.7, while the minority fourteen percent (14%) failed (required detail assessment) with final scores ranging between 1.9 and 2.4. The major contributors to building failure in this community are as a result of vertical irregularity of buildings and the soil type D.

Table 14*:* The summarized performance of structures in the Iterboreale Community (A3A - A3T)

Soil Type	No. of Buildings	Building Type	Plan Irregularity(%)	Vertical Irregularity (%)	Both Vertical & Plan Irregularity (%)	No Irregularity (%)	Properly Constructed (%)	Poorly Constructed (%)	Pass		Failed	
									%	Final scores 'S'	%	Final scores 'S'
C	119	Reinf. Concrete	41	0	9	50	100	0	91	≥ 4.3	9	2.3
	2	Wood	0	0	0	0	50	50	100	≥4.6	0	-
D	63	Reinf. Concrete	19	8	16	57	100	0	76	≥3.9	24	1.9 - 2.4
	12	Wood	25	0	0	75	8	92	100	≥3.7	0	-
	1	W/Concrete	0	0	0	0	0	100	0	-	100	2.0
Overall Performance	**197**								**86**	**≥3.7**	**14**	**1.9 - 2.3**

The community bordered by Iterboreale to the east and the Hospital Road to the west

The community located to the west of Iterboreale and east of the Hospital Road sits only on soil type D. Wood structures reflected very good seismic performance (S-score 3.7- 4.6) while reinforced concrete reflected fairly good performance (S-score 3.9 - 4.4). This is result of the fact that most of these structures did not have any irregularities. On the other hand W/concrete structures proved very detrimental as all (4) existing buildings reflected seismic vulnerability as they were improperly constructed (vertical and plan irregularities) and also mostly old structures (S-score 2.0).

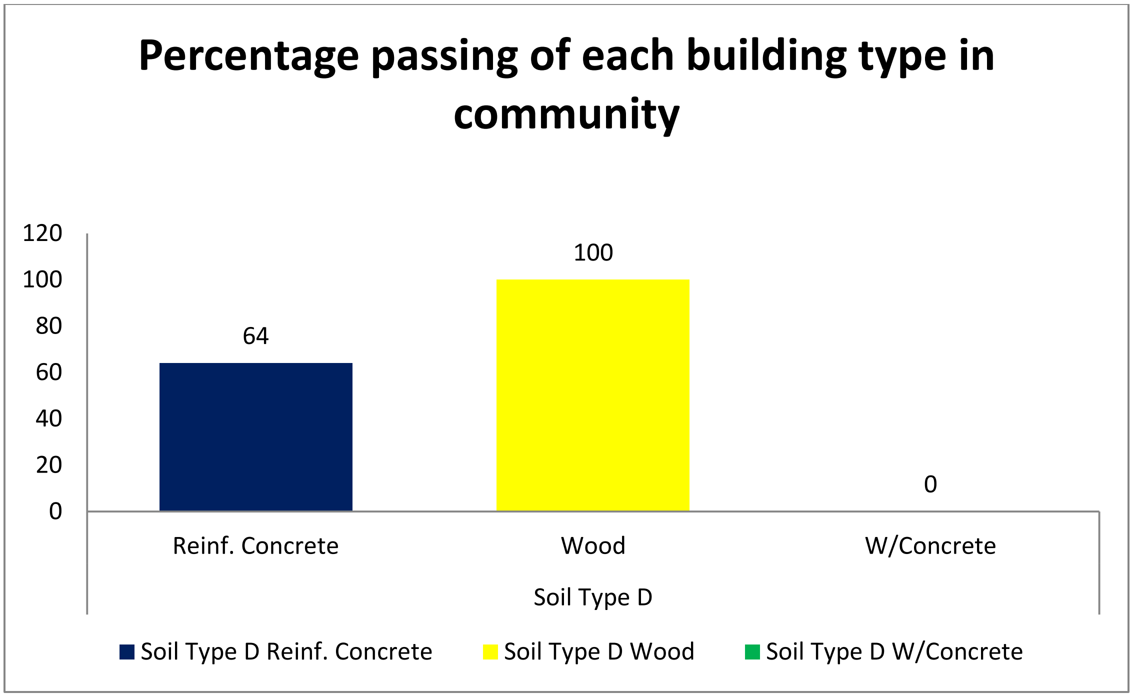


Figure 13: Column chart illustrating percentage passing of each building type when modifiers are applied.

Wood structures are generally better seismic performers than reinforced concrete in soil type D; however, in this instance they produced lower final score. This is due to the fact that most of the wood structures in this vicinity were in deplorable conditions and as such reflected reduced performances, (see Table 15).

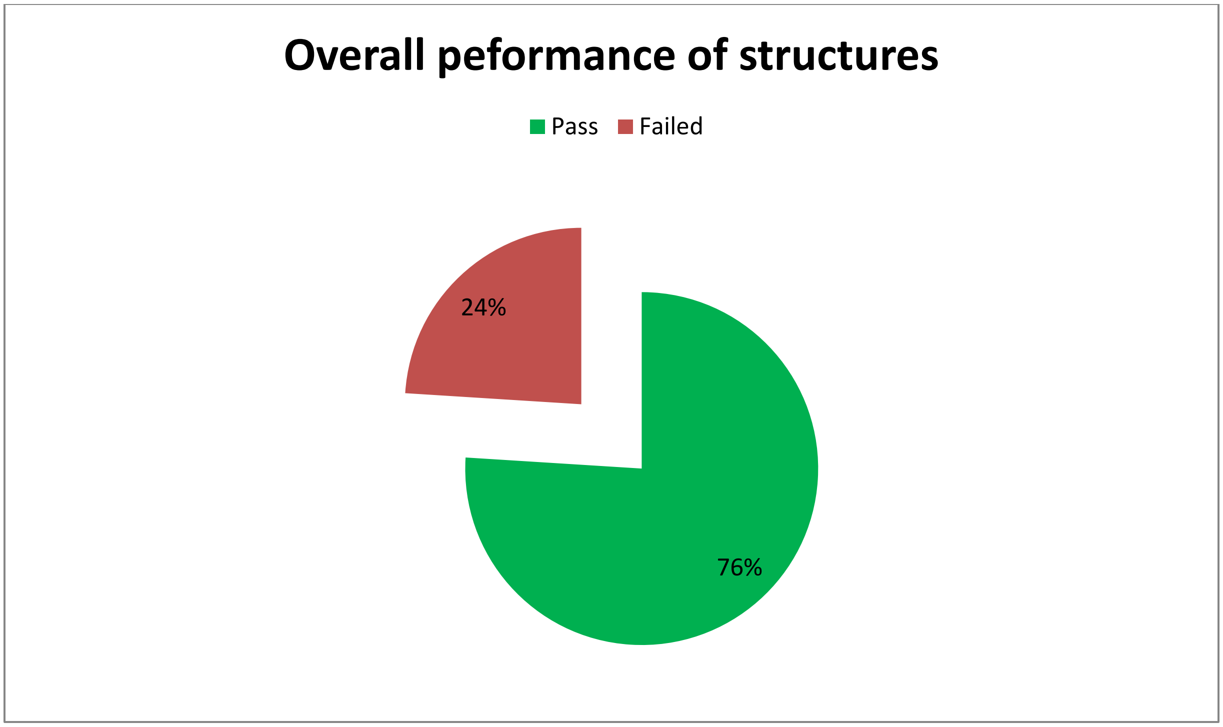


Figure 14: Pie chart illustrating the overall performance of structures in the community

Most of the structures found in this area were reinforced concrete structures (57%) and wood structures (40%); all reflected moderate seismic performance in soil type D due to the absence of vertical irregularities. Therefore, it is seen that seventy six (76%) percent of a total of 122 buildings passed with the lowest final score produced being 3.7. The minority twenty four (24%) percent that failed was a combination of reinforced concrete and W/concrete which produced final scores ranging between 1.9 and 2.4. The major contributors to failure included the soil type (across the board), vertical irregularity (reinforced concrete) and deterioration and improper construction (W/Concrete) structures.

Table 15*:* The summarized performance of structures in community bordered by Iterboreale to the east and the Hospital Road to the west (A3F - A3H)

Soil Type	No. of Buildings	Building Type	Plan Irregularity(%)	Vertical Irregularity (%)	Both Vertical & Plan Irregularity (%)	No Irregularity (%)	Properly Constructed (%)	Poorly Constructed (%)	Pass %	Pass Final scores 'S'	Failed %	Failed Final scores 'S'
D	69	Reinf. Concrete	14	0	14	72	100	0	64	≥3.9	36	1.9-2.4
	49	Wood	10	0	0	90	31	69	100	≥3.7	0	-
	4	W/Concrete	0	0	0	100	0	100	0	-	100	2.0
Overall Performance	**122**								**76**	**≥3.7**	**24**	**1.9 - 2.4**

The community located in the vicinity of the Hospital and the Gibraltar Housing Scheme

The community is situated on both soil type C and D consisting of three building types. Reinforced concrete structures reflected moderate to very good seismic performance producing final scores of at least 3.9 and 4.3 in soil type D and C respectively. However, in circumstances where vertical irregularities were present they reflected seismic vulnerability with reduced performance score of 2.3 in soil type C, as no vertical irregularities were present in soil type D. Additionally, wood structures reflected sound seismic performance both in soil type C and D with scores of at least 4.6 and 3.9 respectively.

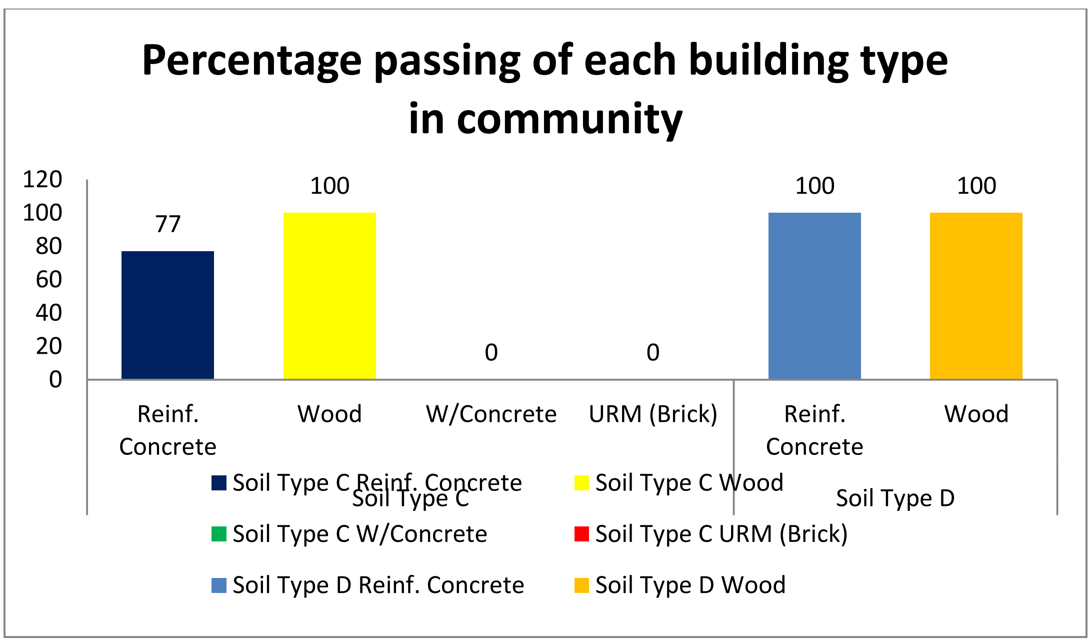


Figure 15: Column chart illustrating percentage passing of each building when modifiers are applied.

Wood structures with vertical irregularities were only found in soil type C and they reflected reduced seismic performance with final scores of 2.6. Unreinforced masonry (URM) and W/Concrete structures were only present in soil type C and they both reflected poor seismic performance as result of age and deterioration (S-score = 2.1) and the presence of vertical irregularities (S-score =1.) respectively. (See Table 16)

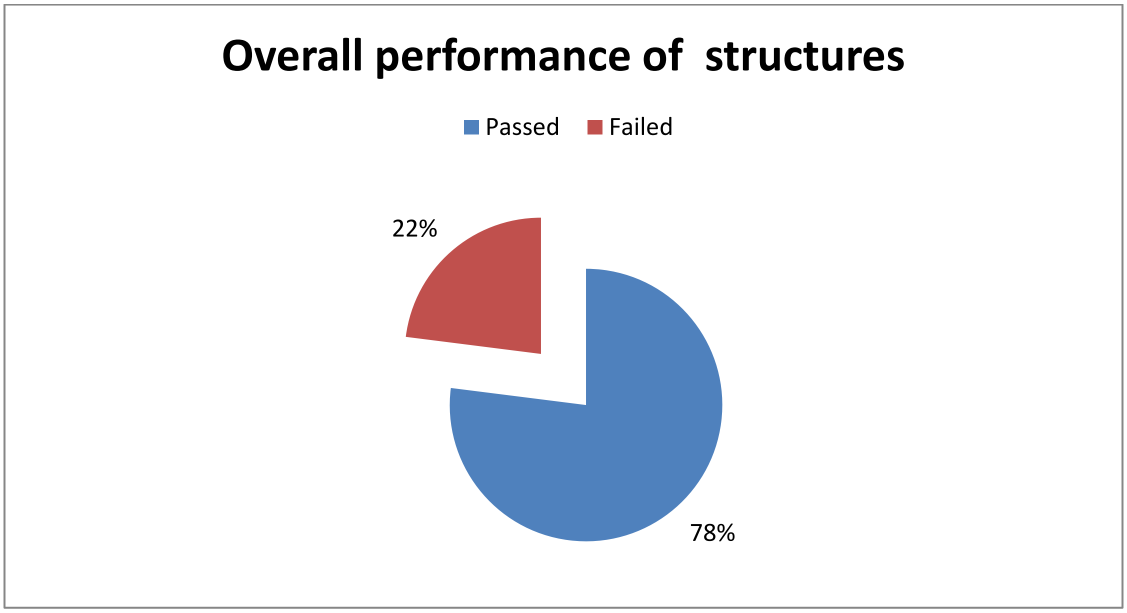


Figure 16: Pie chart illustrating the overall performance of structures in the community.

Most of the structures found in this area (developed housing scheme) were recently constructed reinforced concrete structures which reflected on average good seismic performance in both soil types. Thus, seventy eight percent (78%) of a total of 306 buildings passed with the lowest final score produced being 3.7.The remaining twenty two percent (22%) that failed produced final scores ranging between 1.0 and 2.3 see (Figure 16). The major contributor to failure in soil type C was vertical irregularity (reinforced concrete and W/Concrete) and the pre-code factor (URM).

Table 16: *The summarized performance of structures in the vicinity of the Hospital and Gibralta Housing Scheme (A3H - A3J)*

Soil Type	No. of Buildings	Building Type	Plan Irregularity (%)	Vertical Irregularity (%)	Both Vertical & Plan Irregularity (%)	No Irregularity (%)	Properly Constructed (%)	Poorly Constructed (%)	Pass		Failed	
									%	Final scores 'S'	%	Final scores 'S'
C	278	Reinf. Concrete	24	0	23	53	100	0	77	≥ 4.3	23	2.3
	17	Wood	0	0	6	94	35	65	100	4.6	0	-
	3	W/Concrete	0	33	0	67	100	0	0	-	100	1.1- 2.0
	1	URM (Brick)	100	0	0	0	100	0	0	-	100	2.1
D	4	Reinf. Concrete	75	0	0	25	100	0	100	≥3.9	0	-
	3	Wood	67	0	0	75	67	33	100	≥3.7	0	-
Overall Performance	306								78	≥3.7	22	1.0 - 2.3

The community located between the Gibraltar Housing Scheme Road and Pencar River

This community was located on soil type D, with three building types namely reinforced concrete, wood and W/Concrete, with wood structures reflecting the best seismic performance (S-score 3.7 - 4.6).

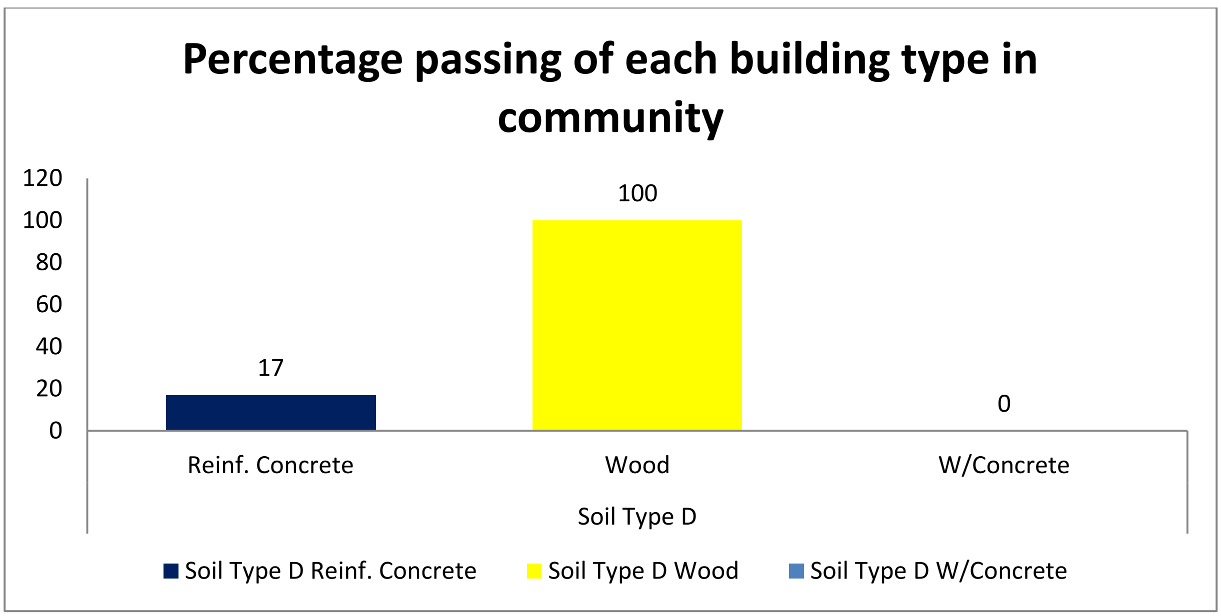


Figure 17*:* Column chart illustrating percentage passing of each building type when modifiers are applied.

Reinforced concrete constituted the larger percentage concentration of buildings but a fair amount of the existing building showed both plan and vertical irregularities which further reduce their scores on this soil type. Additionally, only 17 % of these structures were recently constructed and done so properly producing final scores of 4.4. All building types located on this soil type except W/Concrete structures reflected moderate seismic performance, as these structures were poorly constructed. (See Table 17)

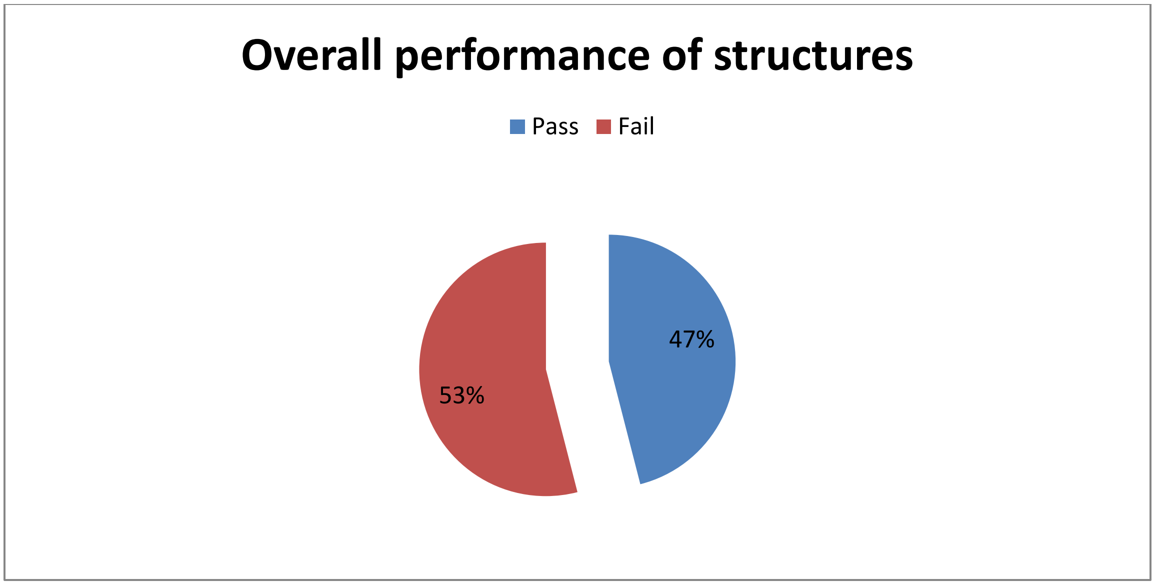


Figure 18: Pie chart illustrating the overall performance structures in the community

Reinforced concrete structures accounted for the largest percentage (62%) concentration of buildings in this area, followed by wood structures at 36%. However, 83% of a total of 84 reinforced concrete structures failed (S-score 1.9 -2.4) and thus negatively affected the overall percentage of that building type that passed. As a result, only 47% of a total of 135 buildings passed with the lowest final score produced being 3.7 .The remaining fifty three percent (53%) that failed produced final scores ranging between 1.9- 2.4 and included both reinforced concrete and W/concrete structures. Major contributors to failure included the soil type (across the board), vertical irregularities (reinforced concrete) and poorly maintained structures (W/Concrete). Complete data assessment forms can be found in Appendix C.

Table 17: *The summarized performance of structures in community from Gibraltar Housing Scheme Road to Pencar River (A3J - Pencar River)*

Soil Type	No. of Buildings	Building Type	Plan Irregularity(%)	Vertical Irregularity (%)	Both Vertical & Plan Irregularity (%)	No Irregularity (%)	Properly Constructed (%)	Poorly Constructed (%)	Pass		Failed	
									%	Final scores 'S'	%	Final scores 'S'
D	84	Reinf. Concrete	25	0	10	65	100	0	17	4.4	83	1.9 - 2.4
	49	Wood	8	0	0	92	31	69	100	≥3.7	0	-
	2	W/Concrete	0	0	0	100	0	100	0	-	100	2.0
Overall Performance	**135**								**47**	**≥3.7**	**53**	**1.9 - 2.4**

The community located between Pencar River and Annotto River

This community is located only on soil type E and consisted of seven building types with reinforced concrete and wood structures accounting for the highest percentage concentration at 48% and 42% respectively. In this soil type only wood and containers (steel) reflected very good seismic performance with 98% (S- score 3.5- 4.0) and 100% (S-score 3.4) passes respectively. The only other building type that had passes was reinforced concrete which had 8% (S-score 1.5-2.0) of a total of 248 RM1 buildings passing due to the absence of irregularities and the application of the post code modifier. Although most of the structures were considered to be built within the post code period the post code modifier was not applied as a fair amount of these structures were observed on squatter like settlements and design and construction methods would imply the absence of adequate engineering considerations and approval. The remaining building types all reflected very poor seismic performance with all the buildings for each category failing with final scores ranging between -0.6 and 2.0. This is due to the fact that the low scores assigned to the soil type, and also some of the structures were deteriorated while others were built within the pre-code era. (See Table 18)

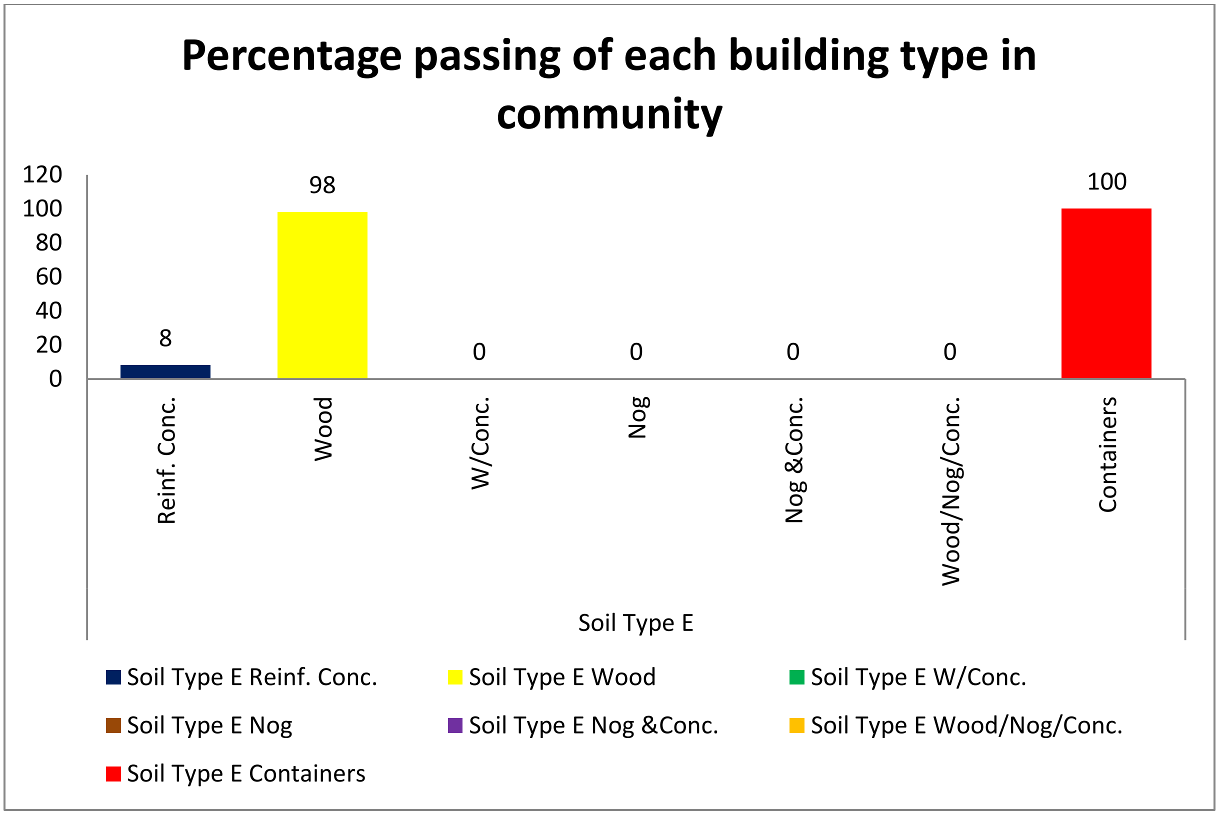


Figure 19: Column chart illustrating percentage passing of each building type when modifiers are applied.

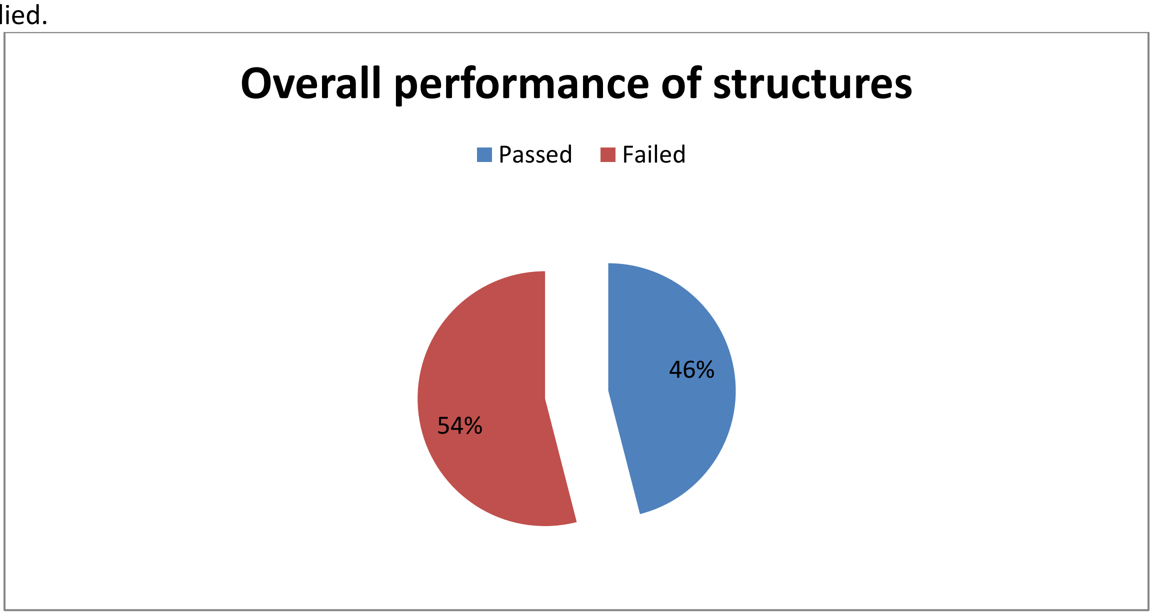


Figure 20: Pie chart illustrating the overall performance of structures in the community

As RM1 buildings accounted for the highest building type concentration (46%) and only eight (8%) percent of these buildings pass the RVS; The poor seismic performance of these structures (see Figure

19) negatively affected the overall percentage of buildings in this community.. Therefore, only 46% of a total of 521 buildings passed with the lowest final score produced being 3.1. The remaining fifty four percent (54%) that failed produced final scores as low as - 0.6. The major contributors to failure included the soil type (across the board), vertical irregularities (RM and all building type combination with RM1) and the pre-code factor.

Table 18: *The summarized performance of structures in community located between Pencar River and Annotto River*

Soil Type	No. of Buildings	Building Type	Plan Irregularity(%)	Vertical Irregularity (%)	Both Vertical & Plan Irregularity (%)	No Irregularity (%)	Properly Constructed (%)	Poorly Constructed (%)	Pass		Failed	
									%	Final scores 'S'	%	Final scores 'S'
E	248	Reinf. Concrete	24	0	5	71	100	0	8	4.0	92	1.5 - 2.0
	216	Wood	16	0	2	82	39	61	98	≥3.5	2	0
	45	W/Concrete	64	0	0	36	67	33	0	-	100	1.5-2.0
	1	Nog	0	0	0	100	100	0	0	-	100	1.8
	1	Nog &Concrete	0	0	0	100	0	100	0	-	100	1.3
	1	Wood/Nog/Conc.	0	0	100	0	100	0	0	-	100	-0.6
	9	Containers	0	0	0	100	0	100	100	3.4	0	-
Overall Performance	**521**								**46**	**≥3.1**	**54**	**-0.6**

The community extending from Annotto River to Fire Station End

This community is located only on soil type E, with four building types namely; reinforced concrete, wood, W/concrete and nog, with percentage concentration being 48%, 41%, 10%,1% respectively. Of the four building types only wood structures reflected sound seismic performance (100% passes) with final scores ranging between 3.5 and 3.6. Conversely, the remaining three building types all had 100% failure with final scores ranging between 1.5 and 2.0. (See Table 19)

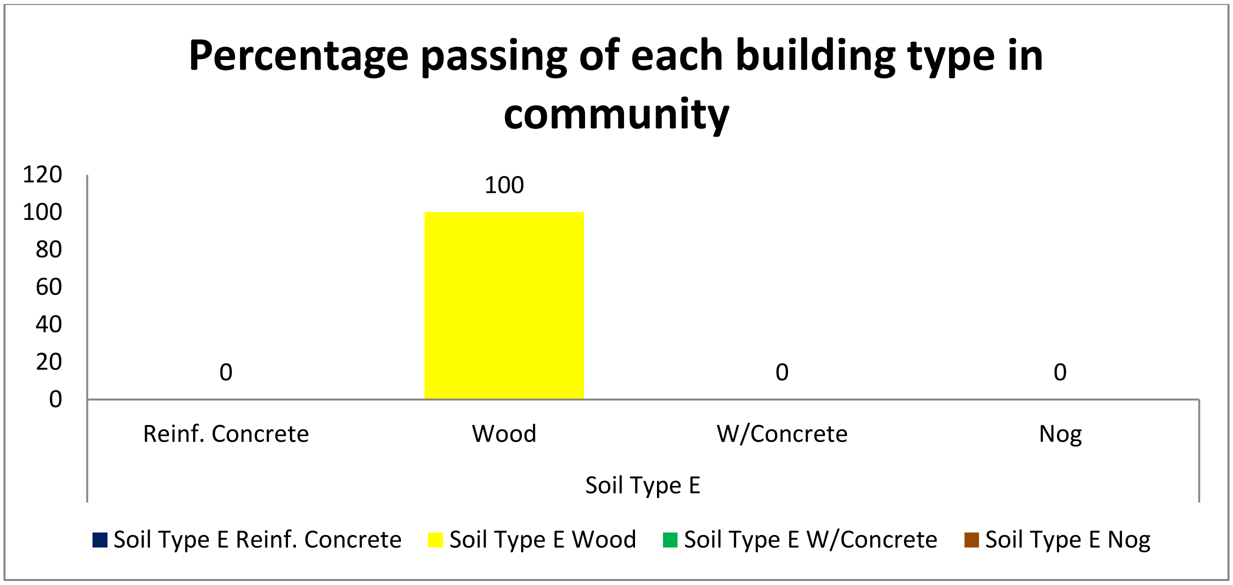


Figure 21: Column chart illustrating percentage passing of each building type when modifiers are applied.

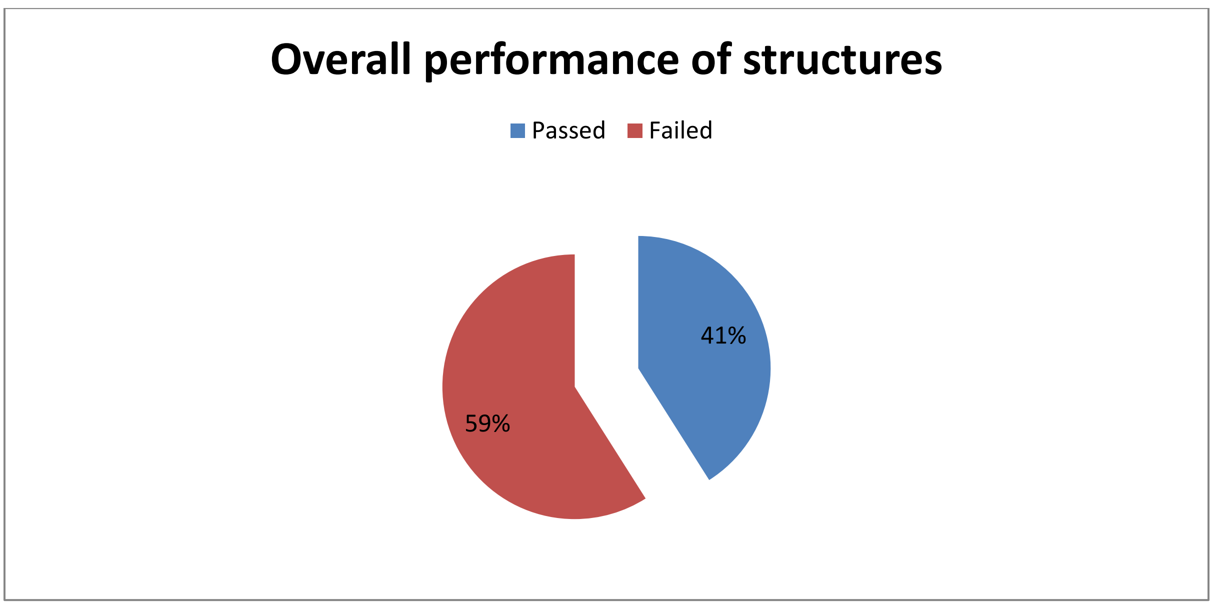


Figure 22: Pie chart illustrating the overall performance structures in the community

As previously mentioned, only wood structures in this community reflected good seismic performance with all passing (100%). As such their 41% concentration was reflected in the overall 41% of buildings that passed with the lowest final score produced being 3.5, the remaining 59% that failed produced final scores between 1.5 and 2.0. The major contributor to failure was the soil type, vertical irregularity (reinforced concrete), plan irregularity and pre-code (W/Concrete) and the pre-code factor (Nog).

Table 19: *The summarized performances of structures in community extending from Annotto River to Fire Station End*

Soil Type	No. of Buildings	Building Type	Plan Irregularity(%)	Vertical Irregularity (%)	Both Vertical & Plan Irregularity (%)	No Irregularity (%)	Properly Constructed (%)	Poorly Constructed (%)	Passed		Failed	
									%	Final scores 'S'	%	Final scores 'S'
E	52	Reinf. Concrete	48	0	13	39	100	0	0	-	100	1.5 - 2.0
	45	Wood	29	0	0	71	11	89	100	≥3.5	0	-
	11	W/Concrete	36	0	0	64	9	91	0	-	100	1.5-2.0
	1	Nog	0	0	0	100	0	100	0	-	100	1.8
Overall Performance	**109**								**41**	**≥3.5**	**59**	**1.5 - 2.0**

The community of Grays Inn

This community is located on soil type D, with five building types namely; reinforced concrete, wood, W/concrete, URM (brick) and wattle and daub, with percentage concentration being 49%, 46%, 3%,1% and 1% respectively. Wood, URM and Wattle & Daub structures were the only building types that reflected good seismic performance giving 100% passes with final scores of at least 2.6. This was a result of 97% of the wood structures and 100 % URM and wattle &daub having no irregularities.

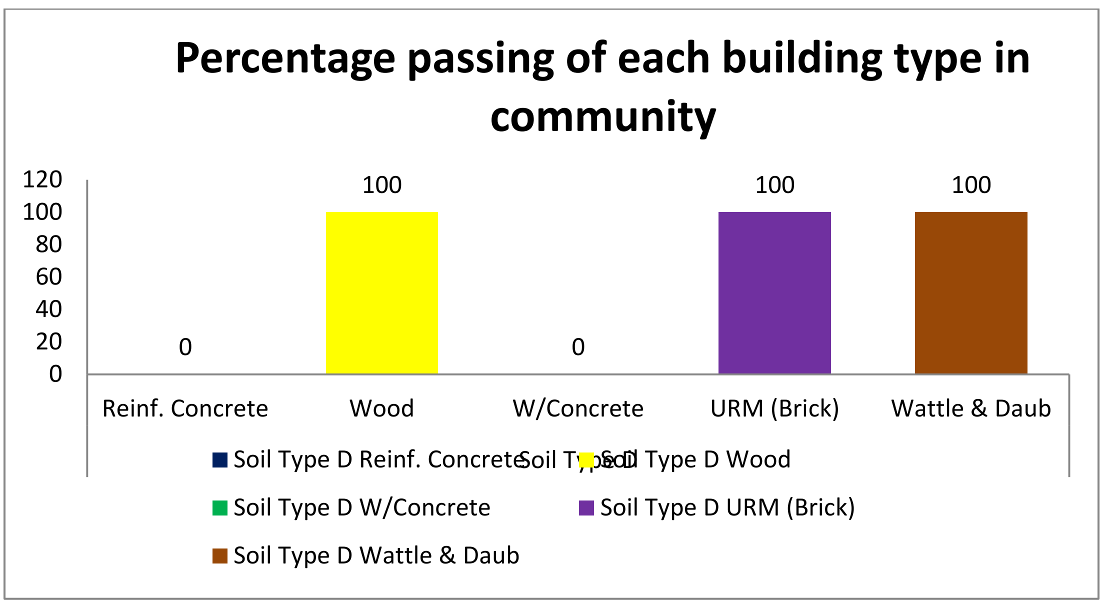


Figure 23: Bar chart illustrating percentage passing of building types based on soil type

Conversely, the remaining two building types all had 100% failure with final scores between 1.5 and 2.4. Apart from the soil type being detrimental on the seismic performance of these structures, most of the reinforced structures were also affected by the pre-code factor while some of the W/Concrete structures were improperly constructed. (See Table 20)

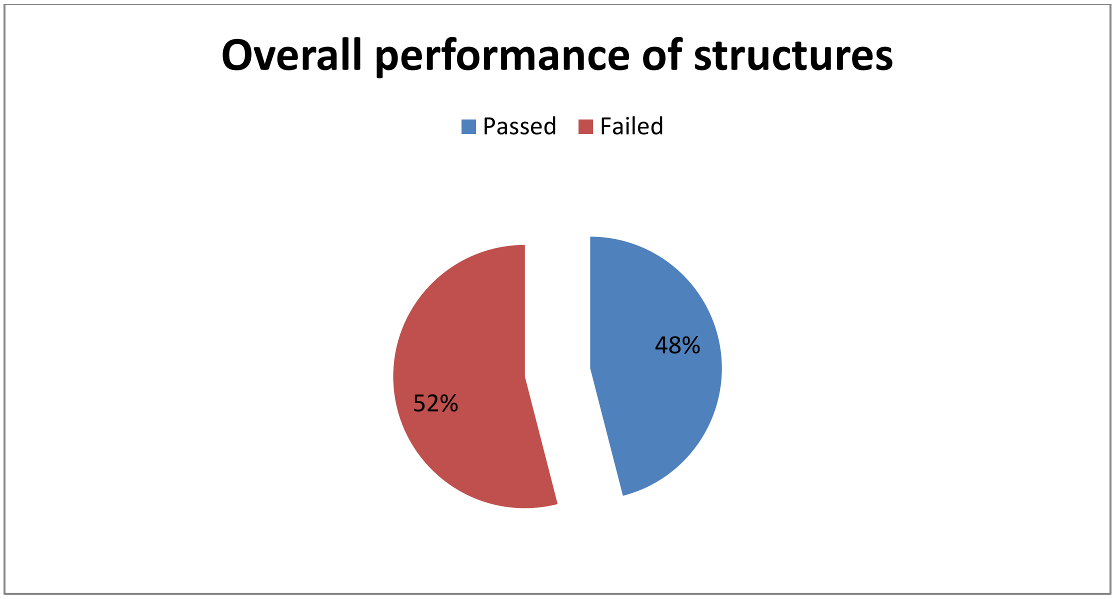


Figure 24: Pie chart illustrating the overall performance structures in the community

As wood, URM and wattle & daub structures were the only building type to produce sound seismic performances (100%), their respective 46%, 1% and 1% percentage concentration determined the overall 48% buildings that passed (S-score 2.6 – 4.6). The remaining 52% that failed comprised of the remaining two building types with final score between 1.5 -2.4. The major contributors to failure included the soil type, deterioration of structures and the pre-code factor.

Table 20: The summarized performance of structures in the community of Grays Inn

Soil Type	No. of Buildings	Building Type	Plan Irregularity(%)	Vertical Irregularity (%)	Both Vertical & Plan Irregularity (%)	No Irregularity (%)	Properly Constructed (%)	Poorly Constructed (%)	Pass		Failed	
									%	Final scores 'S'	%	Final scores 'S'
D	53	Reinf. Concrete	21	0	0	79	100	0	0	-	100	1.9 - 2.4
	50	Wood	4	0	0	96	14	86	100	≥3.7	0	-
	3	W/Concrete	67	0	33	0	67	33	0	-	100	1.5-2.0
	1	URM (Brick)	0	0	0	100	100	0	100	2.6	0	-
	1	Wattle & Daub	0	0	0	100	100	0	100	2.6	0	-
Overall Performance	**108**								**48**	**≥2.6**	**52**	**1.5 - 2.4**

Summary of RVS Results

Overall performance of the building types showed that containers and the wattle & daub (only found in Grays Inn) were the best seismic performers in the study area producing 100% passes; however, it must be noted that these building types only constituted 0.6% and 0.1% of the total number of buildings. The next best seismic performer was wood structures which gave 99% passes out of a concentration of 29.6%. Wood structures have proved to be good seismic performers in all soil types if vertical irregularities are not present, which was mostly the case in this area. Reinforced Concrete structure follows wood structures with 47% of the building passing out of a concentration of 64.8% and then URM structure with 50% passing out of a 0.1% concentration.

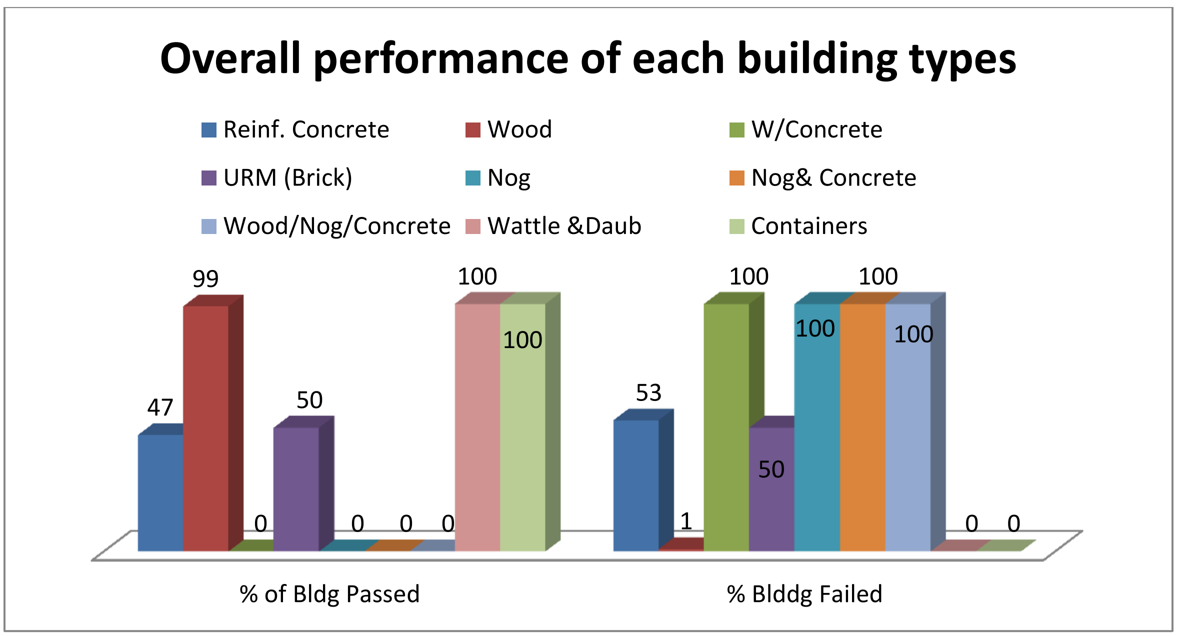


Figure 25: Column chart showing overall performance (% passes and failure) of building types in study area.

Critical Facilities

In the study area, a total of twelve critical facilities, a combination of Emergency, Assembly, Schools and Commercial type occupancy according to FEMA – 154 methodology was screened (See Table 21).

Table 21: The summarized performance of critical facilities in Annotto Bay

Summary of Critical Facilities			
Name	Type of Occupancy	Final Score	Pass/Fail
Annotto Bay All Age	School	≥ 1.6	Fail
Annotto Bay High	School	≥ 2.7	Fail
Annotto Bay Hospital	Emergency	≥ 1.5	Fail
Anntto Bay Court House &Tax Office	Government	≥ 1.4	Fail
Annoto Bay Fire Station	Emergency	3.5	Pass
Annotto Bay Health Centre	Emergency	3.5	Pass
Annotto Bay Library	Government	3.5	Pass
Annotto Bay Police Station	Emergency	≥ - 0.4	Fail
Credit Union	Commercial	4.0	Pass
National Commercial Bank	Commercial	1.5	Fail
St. James Anglican Church	Assembly	≥ 1.5	Fail
St. James Basic School	School	1.5	Fail

The Annotto Bay All Age School consist of three building types RM1, C1 and S1. These structures are located on soil type E and had no irregularities; however, the RM1 and S1 structures were built in the pre-code era. These produced low finals scores of 1.6 (both RM1 and S1) and 2.6 (C1). These had signs of structural defects which also prompted for more detail analysis (see Figure 42 in Appendix 1a).

The Annotto Bay High School consisted of two building types RM1 and C1. These structures were located on soil type D and had plan irregularities. These structures were built within the post-bench mark era; however, signs of major structural defects in the form of major cracks were observed (See Figure 43 in Appendix 1a). These produced least finals scores of 3.9 and 2.7 respectively.

The Annotto Bay Court House &Tax Office was a URM (Nog) structure located on soil type E. This structure was constructed in the pre-code era and had no irregularities. Therefore the final score produced was 1.4.

There were six critical facilities that consisted only of RM1 structures and these included the Annotto Bay Hospital, Fire Station, Library, Heatlh Centre, the National Commercial Bank and the Credit Union. All of these structures except the Hospital (located on soil type D) were located on soil type E. The Hospital buildings had plan irregularities and were constructed in the pre-code era. Additionally, major structural defects were also observed and included major cracks, exposed reinforcement and one observed dislocation of a column from a building due to soil movement. Final scores that were produced by these structures ranged from 1.5 and 2.0. (See Figure 44 in appendix 1a). The Fire station, Library and the Health centre were all built in the post-benchmark era and had only plan irregularities. Therefore the final score produced by all these structures was 3.5. However, the Credit Union had neither plan nor vertical irregularities and was also produced in the post-benchmark era. This building produced a final score of 4.0 which was the highest of all the critical facilities. The National Commercial Bank had both plan and vertical irregularities and these factors (especially vertical irregularity) along with the soil factor are very detrimental to seismic performance and as such produced a final score of 1.5.

The St. James Anglican Church and Basic school were both located on soil D and were constructed in pre-code era. The Church was combination of RM1 and URM (Brick) and had plan irregularities and thus produced final scores of 1.5 and 1.7 respectively. On the other hand, the basic school was only RM1 and included plan irregularities and produced a final score of 1.5.

The Annotto Police Station was the facility that had more than two building types and these included RM1, URM (Brick) and W1. This facility was located on soil type E and included both plan and vertical irregularity. This facility produced the lowest final scores of all the critical facilities and included 1.5, -0.6 and -0.4 for RM1, URM (Brick) and W1 respectively.

Finally, out of the twelve critical facilities that were screened only four (approximately 33%) of these structures passed (no detailed assessment is necessary), while the remaining eight (approximately 67%) failed (detailed assessment is necessary). The least final scores produced for passes and failures were 3.5 and -0.6 respectively; see figure 26.

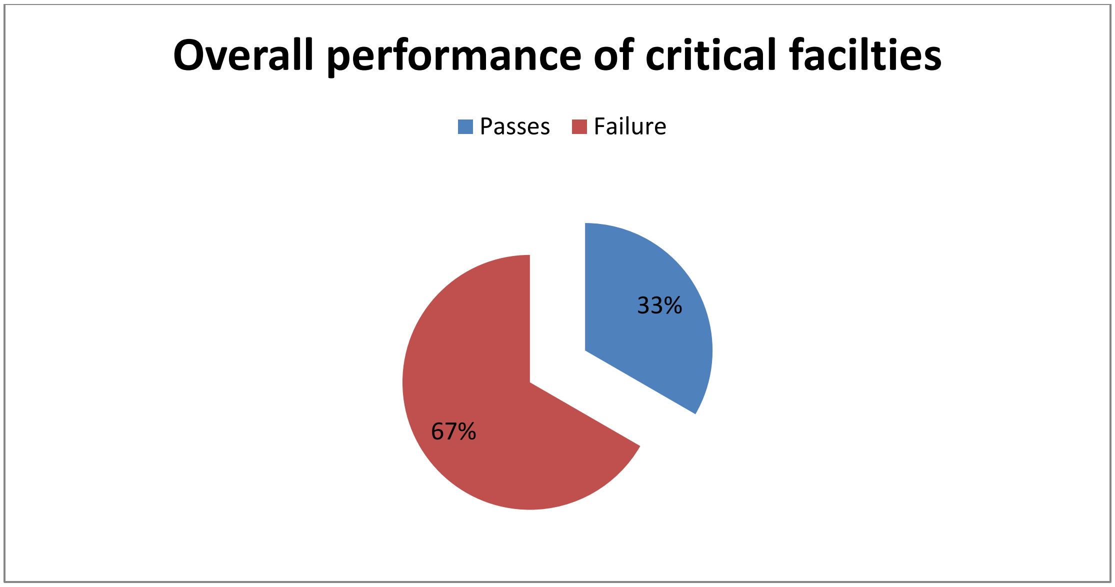


Figure 26: Pie chart showing overall performance (% passes and failure) of critical facilities in study area.

Table 22: The distribution of buildings in the community and the percentage of structures passing based on the RVS.

Community	**Number of Buildings**	**Percent passing based on RVS**
Iterboreale	197	86
Community between Iterboreale and Hospital	122	76
Vicinity of Hospital and Gibraltar Housing Scheme	306	78
Gibraltar Housing Scheme to Pencar River	135	47
Pencar River to Annotto River	521	46
Annotto River to Fire Station	109	41
Community of Grays Inn	108	48

Based on this data (table 22), the decision was taken to zone the town of Annotto Bay based on the results of the Rapid Visual Screening. Zone 1 include between 80-100 % of building passing the cut-off score and is designated as passing. Zone 2 include between 60-80% of the buildings passing, the Zone 3 has the lowest scores with only 40-60 % of buildings passing the RVS.

This zoning scheme can be used as a means of demarcating the town into the seismic vulnerability based on the quality of the buildings in the town. Figure 27 shows demarcation of Annotto Bay into these vulnerability zones:

Zone 1: 80-100% passing

Zone 2: 60-80% passing

Zone 3: 40-60% passing

Appendix C- shows the result of individual assessment of typical structures in the community of Annotto Bay

Conclusion

Based on the H/V assessment there is a clear indication that sections of the town of Annotto Bay should show high ground amplification during a major earthquake, the section in the downtown area close to the police station courthouse are areas where highest amplification is expected and also show characteristics of deep soil thickness. These are potentially unstable areas and as such any building design in these areas must take into consideration the potential for resonance.

As illustrated in Figure. 15, reinforced concrete structures had the largest percentage concentration (64.8%) of buildings and 41%, 28% and 31% of these structures where located on soil type C, D and E respectively. Additionally, it was also found that reinforced concrete structures produced poor seismic performance in soil type D (especially if vertical irregularity was present) and very poor seismic performance in soil type E. This was reflected in the 60% and 93% failure that were produced in soil type D and E respectively. On the other hand, these structures performed very well in soil type C (vertical irregularity absent) with 81% of the structures passing.

Even though most of the W1 structures are in poor conditions they still performed well as wood is generally a good seismic performer in any soil type, provided no vertical irregularities are present.

Since soil type C had the most reinforced concrete structures and also had very good seismic performance, this and the remaining passes from soil type D and E resulted in reinforced concrete producing 30.2% of the total amount of buildings that passed. Wood structures that had the second highest concentration of buildings (29.6%) proved to be very good seismic performers in all soil types given that vertical irregularity was not present. They produced 99% passes thus contributing to 29.3% of the overall total of buildings that passed. The only other building types that produced passes were URM (brick), wattle & daub structures and containers and they made a 0.7% contribution to the overall total of buildings that passed. Therefore, 60.2% of the buildings in Annotto Bay would produce adequate seismic performance (did not require detailed assessment) and 39.8% would not produce sufficient seismic performance (needed detailed assessment) in the event of an earthquake (See Figure 28). Likewise it must be noted that only 33% of the overall critical facilities that were present in the study area would produce adequate seismic performance in similar circumstance while 67% would not.

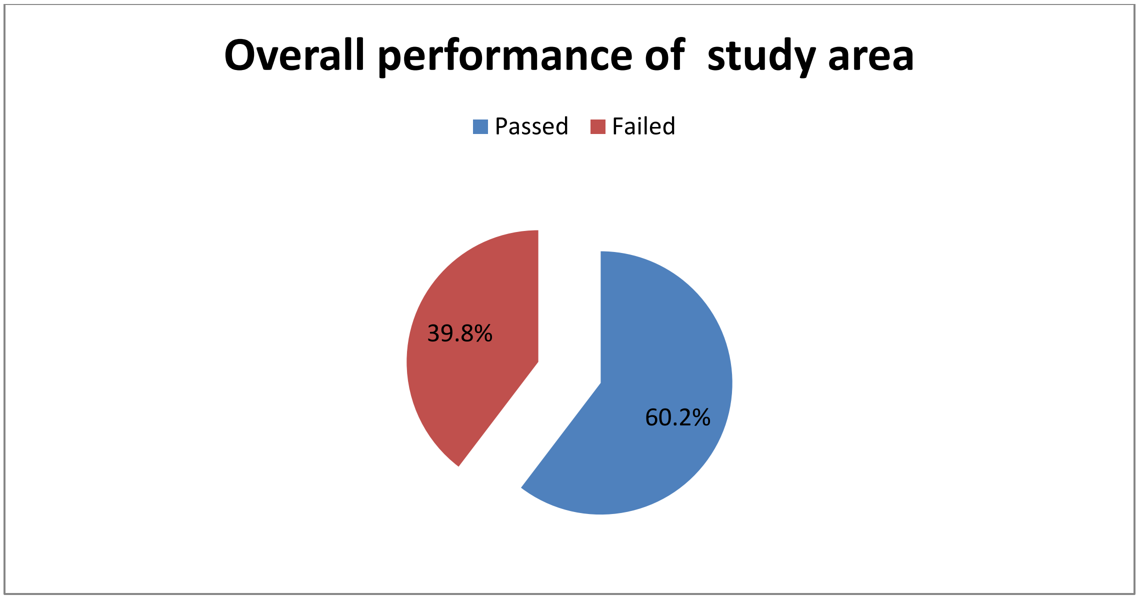


Figure 28: Pie chart showing overall performance (% passes and failure) of structures in study area.

A more detailed assessment is required for this coastal community to see if evidence still in the shallow coastal waters that can give evidence of previous tsunami activity and coastal fault systems.

Figure 27, give strong guidance of the potential for ground amplification in sections of the town and also provides guidance in terms of future building development and the correlation with potential for resonance of structures

Table 23*:* The overall performance of building types in Annotto Bay

Overall performance of building types in Annotto Bay						
Building Type	**No. of Bldgs**	**Concentrated % of Bldg types**	**No. of Bldgs Pass**	**No. of Bldgs Failed**	**% of Bldgs Passed**	**% of Bldgs Failed**
Reinf. Concrete	970	64.8	452	518	47	53
Wood	443	29.6	439	4	99	1
W/Concrete	69	4.6	0	69	0	100
URM (Brick)	2	0.1	1	1	50	50
Nog	2	0.1	0	2	0	100
Nog& Concrete	1	0.1	0	1	0	100
Wood/Nog/Concre	1	0.1	0	1	0	100
Wattle &Daub	1	0.1	1	0	100	0
Containers	9	0.6	9	0	100	0
Total	**1498**	**100**	**902**	**596**		

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Appendix A

Figure A1: A group of pictures showing major defects to the Annotto Bay All-Age School, in the form of pop-outs and exposed reinforcement. Due to the exposure of these bars they have begun to rust and over-time will increase the risk of failure.

Figure A2: A group of pictures showing major defects to the Annotto Bay High School, in the form of major cracks in vital structural members (Beam) and also pop-outs. There is also evidence of water contact which suggests that reinforcements are being damaged (rust) which cause swelling of the bars, which reflect pop-outs and cracks.

Figure A3: A group of pictures showing defects to the Annotto Bay Hospital in the form of major cracks and the moving away of a column from the building due to the presence of soil movement, which actually left large cracks in the ground.

Appendix B

Table B1: Building types within the community of Iterboreale along with building descriptors

The Community of Iter Boreale									
Street Name	Soil Type	No. of Buildings	Building Type	Storeys Two	Storeys Three	Plan Irregularity	Vertical Irregularity	Properly Constructed	Poorly Constructed
A3A (Start point Scheme 1 frm Dry River) - A3BS	C	31	Reinf. Concrete	1	-	6	1	Yes	-
		2	Wood	-	-	-	-	1 Board	1 Plywood
A3BN	D	7	Reinf. Concrete	1	-	2	1	Yes	No
		1	Wood	-	-	-	-	No	1 Plywood
		1	W/Concrete	-	-	-	-	No	Deteriorating board Section
A3BS-A3CS	C	13	Reinf. Concrete	1	-	5	1	Yes	No
A3CN	D	7	Reinf. Concrete	6	1	1	6	Yes	No
		2	Wood	-	-	-	-	1 Board	1 Plywood
A3CS (Scheme 2)	C	74	Reinf. Concrete	10	-	48	8	Yes	1 Visible major crack in wall
A3CS - A3D (both sides of main)	D	28	Reinf. Concrete	8		20	7	Yes	No
A3D - A3T (pass High School opposite side of main at Cellular Tower)	D	22	Reinf. Concrete	7	-	5	2	Yes	1 buildign with major cracks
		9	Wood	-		2	-	-	1 deterioirating board; 6 Plywood
Total		197		34		89	26		

Table B2: **Building types within the community bordered by Iterboreale and the Annotto Bay Hospital along with building descriptors**

The Community that is bordered by Iter Boreale to the left and Hospital Road to the Right									
Street Name	Soil Type	No. of Buildings	Building Type	Storeys Two	Storeys Three	Plan Irregularity	Vertical Irregularity	Properly Constructed	Poorly Constructed
A3F-A3G (Cricket River)	D	10	Reinf. Concrete	2	-	6	2	Yes	No
		24	Wood	-	-	4	-	7 Board	14 plywood; 3 board
		2	W/Concrete	-	-	-	-	Yes	1 Deteriorating board Section
A3G	D	37	Reinf. Concrete	5	-	5	5	Yes	No
		10	Wood	-	-	-	-	3 Board; 1 plywood	1 Deteriorating board; 5 plywood
A3G- A3H (Hospital Rd)	D	22	Reinf. Concrete	7	-	9	3	Yes	No
TableB3:		15	Wood	-	-	1	-	4 Board	5 plywood; 6 deteriorating board
		2	W/Concrete	-	-	-	-	-	Cracks in concrete sections and deterirorating board sections
Total		122		14		25	10		

Building types within the community bordered by Annotto bay Hospital and the Gibraltar Housing Scheme along with building descriptors

The Community in the vicinity of the Hospital and the Gibralta Housing Scheme									
Street Name	Soil Type	No. of Buildings	Building Type	Storeys		Plan Irregularity	Vertical Irregularity	Properly Constructed	Poorly Constructed
				Two	Three				
A3H (Hospital Rd) -A3J (Gibralta Housing Scheme Entrance)	D	4	Reinf. Concrete	-	-	3	-	Yes	No
		3	Wood	-	-	3	-	2 Board	1 Deteriorating board
A3J (Gibralta Housing Scheme)	C	278	Reinf. Concrete	58	3	131	64	Yes	13 with major cracks; 1 not properly constructed
		17	Wood	-	-	1	1	2 plywood and the remainding are board	4 Deteriorating board; 7 Plywood
		3	W/Concrete	1	-	-	1	Yes	No
		1	URM (Brick)	-	-	1	-	Yes	No (but architectual design signifies it is a very old building)
Total		306		59	3	139	66		

Table B4: **Building types within the community between Gibralta Housing Scheme Road and Pencar River along with building descriptors**

The Community located between the Gibralta Housing Scheme Road and Pencar River									
Street Name	Soil Type	No. of Buildings	Building Type	Storeys		Plan Irregularity	Vertical Irregularity	Properly Constructed	Poorly Constructed
				Two	Three				
A3J - Pencar River	D	84	Reinf. Concrete	22	-	29	8	Yes	No
		48	Wood	-	-	4	-	4 plywood the remainding are board	21 plywood; 10 deteriorating board
		2	W/Concrete	-	-	-	-	Yes	No
Total		134		22	-	33	8		

Table B4: **Building types within the community bordered by Pencar River and Annotto River along with building descriptors**

The Community located between Pencar River and Annotto River									
Street Name	Soil Type	No. of Buildings	Building Type	Storeys		Plan Irregularity	Vertical Irregularity	Properly Constructed	Poorly Constructed
				Two	Three				
Pencar River - Annotto River	E	248	Reinf. Concrete	37	3	73	13	Yes	3 deterioirating structure; 2 buildigns with major cracks
		1	Wood/Nog/Reinf. Concrete	-	-	1	1	Yes	No
		216	Wood	4	-	39	4	12 plywood; remainder board	97 deteriorating board ; 35 plywood
		45	W/Concrete	-	-	29	-	Remainder in good condition	9 deteriorating board section; 2 deteriorating concrete section; 4 deteriorating plywood section
		1	Nog	-	-	-	-	Yes	No
		1	Nog & Concrete	-	-	1	-	-	Cracks in structure
		9	Containers	-	-	-	-	Yes	No
Total		521		41	3	143	18		

Table B5: **Building types within the community bordered by Annotto River and the Fire Station along with building descriptors**; ;

The Community extending from Annotto River to Fire Station End									
Street Name	Soil Type	No. of Buildings	Building Type	Storeys Two	Storeys Three	Plan Irregularity	Vertical Irregularity	Properly Constructed	Poorly Constructed
Annotto River - Fire Station End	E	52	Reinf. Concrete	8	-	32	7	Yes	No
		45	Wood	-	-	13	-	1 plywood; 4 board structure	23 Deteriorating board; 12 plywood
		11	W/Concrete	-	-	4	-	1 with board section	6 deteriorating wood section; 1 deteriorating plywood section
		1	Nog	-	-	-	-	-	
Total		109		8	-	49	7		

Table B6: **Building types within the community of Grays Innwith the applied modifiers used in the RVS along with building descriptors**

The Community of Grays Inn									
Street Name	Soil Type	No. of Buildings	Building Type	Storeys Two	Storeys Three	Plan Irregularity	Vertical Irregularity	Properly Constructed	Poorly Constructed
Grays Inn	D	53	Reinf. Concrete	1	-	11	-	Yes	6 deteriorating structure (major cracks)
		3	W/Concrete	1	-	2	1	Yes	1 deterirating structure
		50	Wood	-	-	2	-	6 board	26 deteriorating plywood; 15 deteriorating board
		1	Brick	-	-	-	-	-	Very old Structure
		1	Wattle and daub	-	-	-	-	Yes	No
Total		108		2	-	15	1		