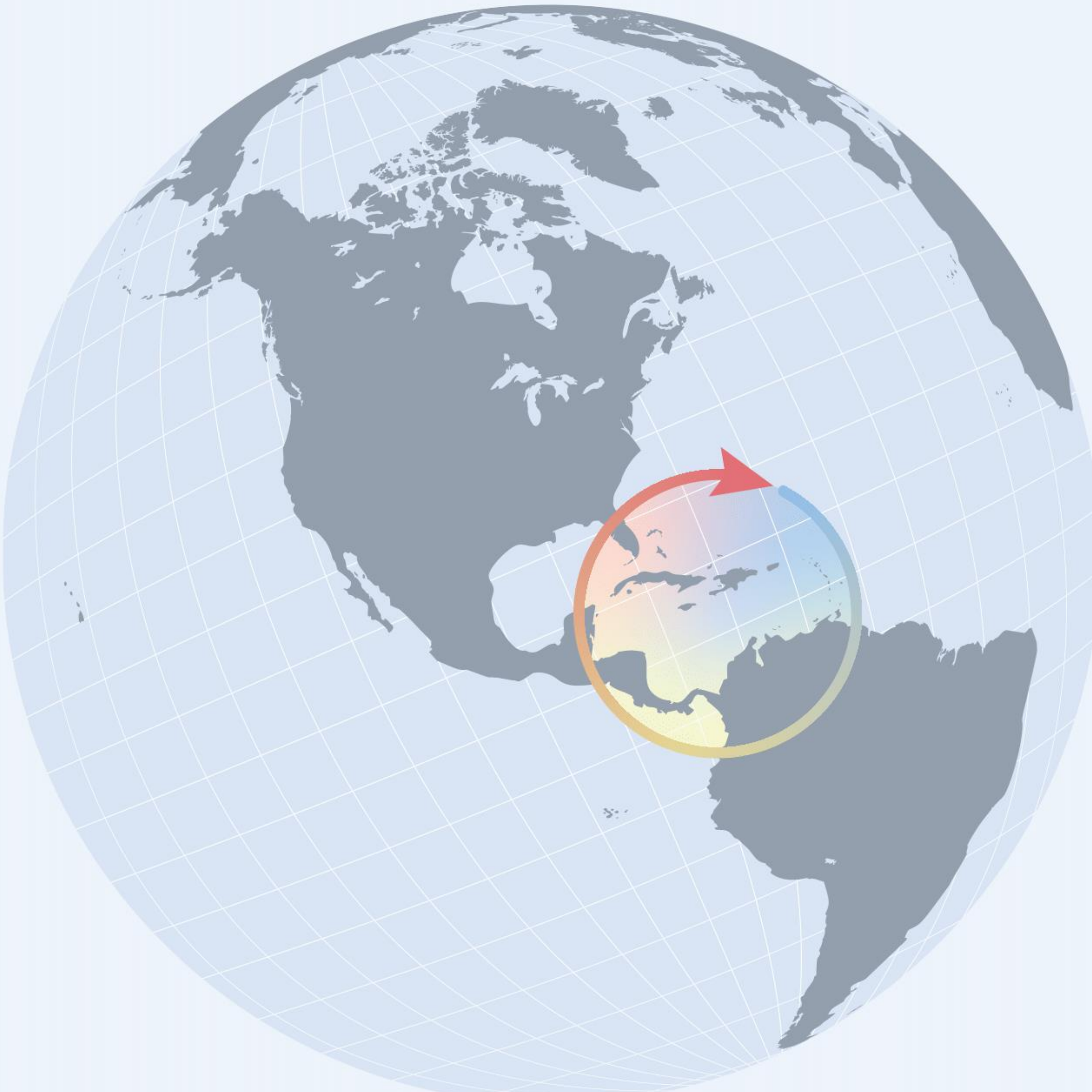
A BACKGROUND PAPER >> ENGINEERING OPTIONS FOR RESILIENT INFRASTRUCTURE

360° Resilience

A Guide to Prepare the Caribbean for a New Generation of Shocks



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Overview of Engineering Options for Increasing Infrastructure Resilience in the Caribbean

FINAL REPORT

[Revised] Submitted February 26, 2021
by Miyamoto International, Inc.

Contract 7195879

THE WORLD BANK

miyamoto.

About the Project

Contract #7195879, Overview of Engineering Options for Increasing Infrastructure Resilience in the Caribbean, presents a high-level analysis of the engineering options available to increase the resilience of various infrastructure assets to floods, winds, and earthquakes as well as an estimate of the cost of these options in Caribbean countries.

About the World Bank Group

The World Bank Group is one of the world's largest sources of funding and knowledge for developing countries. Its five institutions share a commitment to reducing poverty, increasing shared prosperity, and promoting sustainable development. The World Bank Group is committed to open development and has opened its data, knowledge, and research to foster innovation and increase transparency in development.

About Miyamoto International, Inc.

Miyamoto International Inc. ("Miyamoto") is a global structural engineering and disaster-risk reduction firm providing resiliency expertise that sustains industries and safeguards communities around the world. We are experts in earthquake-resilient engineering practices that reduce damage and facilitate disaster recovery. We design new construction and assess existing buildings to address specific vulnerabilities to disasters. We prioritize solutions that limit damage, business interruption and loss of life. Miyamoto is strategically located worldwide in regions vulnerable to disaster to positively impact economies and save lives.

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The opinions, findings, and conclusions stated herein are those of the authors (Miyamoto International, Inc.) and do not necessarily reflect the views of the World Bank Group.

[Revised]

Submitted on:

February 26, 2021

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EXECUTIVE SUMMARY

The geographical location of Caribbean counties makes them particularly vulnerable to natural hazards; see Table 1-1 (ThinkHazard, 2020). Buildings, water, transport, and power infrastructure represent key assets for these countries, yet many of these assets are located in highly hazard-prone zones and thus are exposed to the resulting hazard impacts.

Country	Earthquake	Hurricane	Flood (Coastal and river)
Suriname	Low	Low	High
Trinidad and Tobago	Medium	High	High
Guyana	Low	Low	High
Belize	Medium	High	High
Haiti	Medium	High	High
Dominican Republic	Medium	High	High
Antigua and Barbuda	Medium	No data	Medium
Dominica	Medium	No data	High
Grenada	Medium	No data	Medium
Saint Kitts and Nevis	Medium	No data	Medium
Saint Lucia	Medium	No data	Medium
Saint Vincent and the Grenadines	Medium	No data	Medium
Sint Maarten	No data	No data	No data
Barbados	Medium	No data	Medium
Jamaica	Medium	High	High
Bahamas	Medium	High	High

Table 1-1. Relative importance of various hazards in different countries (ThinkHazard, 2020)

The continuous operation of critical public buildings, water, transport, and power infrastructure allows for ongoing economic growth and societal wellbeing. Damage to these sectors results in direct economic losses, operation interruption, and has cascading effects on other infrastructure. To enhance the resiliency of this critical infrastructure, it is important to undertake an evaluation program (condition assessment), identify the vulnerable components, and devise effective strengthening techniques for vulnerable components; the goal of such a program is a reduction in damage, economic losses, and downtime in future natural hazard events.

Table 1-2 presents the key infrastructure, typologies, and vulnerable components considered in this report. The table also identifies the expected governing hazards for various components in the Caribbean. In Table 1-2, the governing hazards include earthquake ground shaking ("earthquake"), earthquake liquefaction ("liquefaction"), hurricane ("wind"), and flood ("flood"). Table 1-3 presents the proposed strengthening measures for the vulnerable components identified in Table 1-2. Also presented in this table are the estimated capital costs and associated benefits with risk reduction once strengthening (mitigation) has been completed. These values are expressed as a ratio relative to the cost of new construction for the assets. The cost-effectiveness of mitigation, as presented in Table 1-3, is consistent with findings from other researchers[1] and can be seen graphically in Figure 1-1, which shows that once retrofitting is implemented, the probability of the infrastructure experiencing severe damage or collapse is reduced from approximately 80% to 20%.

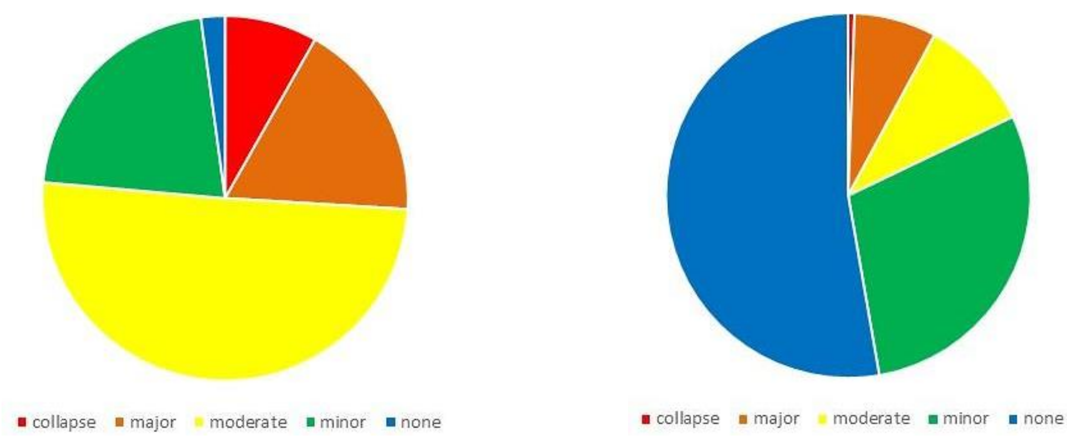
Based on the literature review and findings in this report, the following is noted:

[1] NIBS (2019) shows a benefit-to-cost ratio of 3-to-8 for infrastructure for earthquake, wind, or flood risk mitigation.

- The 16 countries in focus in this report are geographically located in areas that experience a number of natural hazards, which could damage buildings, water, transport, and power infrastructure. Consequences include not only direct structural damage, but also secondary economic losses due to loss of revenue from inoperability, delayed recovery, and cascading costs to other infrastructure.
- Efficacious and economically-feasible strengthening options are available. These measures rely on well-proven techniques and are suitable to the Caribbean countries, given the availability of material and skilled labor resources. Once implemented, such measures would reduce infrastructure vulnerability by multiples of the initial capital cost.
- The service life of infrastructure is comprised of planning, design, construction, maintenance, and post-event monitoring. Devising and implementing a construction quality management plan would ensure that retrofits are built as designed and would add to the service life of infrastructure, reducing the need for corrective maintenance. Preventive maintenance and inspection would reduce the cost of expensive future repairs. Early-warning and post-event inspection can result in reduced delays and expedite the return to functionality.
- Creative financing measures and life-cycle cost analyses (LCCA) can be used to identify and mitigate project risks, accelerate project delivery, and coordinate the design and construction phases.

The following steps are recommended to assist in implementation of an infrastructure resilience program:

- Perform a review of local design and construction codes and prepare recommendations for revisions based on recent intensity of natural hazard events.
- Conduct inventory assessment to develop a searchable database of critical components and to screen the components and facilities for a phased retrofitting program.
- Implement a feasibility study for a selected sector and subsector. Preform probabilistic LCCA to identify the most effective retrofits. For the probabilistic LCCA, incorporate realistic initial capital costs and recurring costs.
- Program and undertake retrofitting for one of the countries in a selected sector. This program can serve as a pilot program for use in other countries with similar regional characteristics.
- Region-wide organizations, like CDEMA, can develop a post-event damage assessment program, prepare paper and electronic forms, train engineers to perform assessments, and maintain an updated database of trained professionals that can perform these tasks in the event of natural hazards.
- Because hurricanes and coastal floods are a major natural hazard for most countries in the Caribbean, undertake an evaluation of flood protection (levees, embankments, etc.) and flood prevention (drainage, collection, etc.) structures in place for coastal areas. Consider improvements and resilience measures for these structures. Such measures provide an additional layer of protection against natural hazards for the infrastructure sectors considered in this report.



Existing Risk mitigated

Figure 1-1. Risk reduction as a result of mitigation

Sector	Subsector	Type	Component	Vulnerability	Hazard(s)
BUILDINGS	Schools Hospitals Government Emergency Hospitality Hotels	Unreinforced Masonry (URM)	Wall	Out-of-plane capacity	Earthquake
				In-plane capacity	Earthquake
			Roof	Roof connection	Wind
		Reinforced/concrete frame (RC/CF) with infill wall	Shape and figure	Structural irregularity	Earthquake
			Wall	Infill wall connection	Earthquake
			Roof	Roof connection	Wind
			Opening	Building envelope	Flood
		RC frame	Shape and figure	Structural irregularity	Earthquake
			Ductility	Non-ductile detailing	Earthquake
			External wall	External wall	Wind
		Steel frame	Stiffness	Stiffness	Earthquake; Wind
			Strength	Structural capacity	Earthquake
			Roof	Roof connection	Wind
		Steel frame high-rise	Stiffness	Stiffness	Earthquake; Wind
			Ductility	Non-ductile detailing	Earthquake
			Cladding	External curtain wall	Wind
		Wood frame	Roof	Roof connection	Wind
			Foundation	Foundation ties	Wind; Flood
			Elevation	Elevated construction	Flood
		All	Foundation	Lack of capacity	Liquefaction

WATER	Water Treatment Plants (WTP) Wastewater Treatment Plants (WWTP)	Plant building (Masonry/RC frame)	Wall	Out-of-plane capacity	Earthquake
				In-plane capacity	Earthquake
			Ductility	Non-ductile detailing	Earthquake
			Roof	Roof connection	Wind
			External wall	External wall	Wind
			Opening	Building envelope	Flood
			Elevation	Elevated construction	Flood
		Water storage	Foundation	Foundation	Earthquake
			Anchorage	Anchorage	Earthquake
		Equipment (mechanical and electrical)	Foundation	Foundation tie	Earthquake; Wind
			Elevation	Elevating equipment	Flood
			Floodproofing	Dry floodproofing	Flood
		All	Foundation	Inadequate capacity	Liquefaction
	Underground Pipelines (UGP)	Pipe	Brittle pipes (Asbestos-cement, cast iron, etc.)	Brittle pipes	Earthquake; Liquefaction; Flood

Sector	Subsector	Type	Component	Vulnerability	Hazard(s)
		Joints	Conventional	Limited deformation capacity	Earthquake; Liquefaction
TRANSPORT	Paved roads	Asphalt paved	Subsurface	Substrate washout	Flood
			Surface	Strength loss	Flood
	Bridges	Concrete girder	Columns	Lack of ductility	Earthquake
			Joints	Short seat	Earthquake
		Steel truss	members	Slender members	Earthquake
			Connections	Riveted connections	Earthquake
		Steel girder	Bearings	Unseating	Earthquake
			Cross frames	Slender members	Earthquake
		Steel bridges	Welded connections	Low fatigue capacity	Wind
		Suspension	Bridge stability	Low torsional stiffness	Wind
		All	Foundation	Lack of capacity	Earthquake
POWER	Power plants (traditional)	Low-rise RC	Design	Nonductile construction	Earthquake
			Grade construction	No flood protection	Flood
			Nonstructural components	Lack of anchorage	Earthquake
			Foundation	Shallow	Earthquake; Liquefaction
	Substations	Control building	Design construction	Strength stiffness	Earthquake
			Grade construction	No flood protection	Flood
			Battery racks, cable trays, etc.	Lack of anchorage/bracing	Earthquake
			Foundation	Shallow	Earthquake; Liquefaction
		Transformers	Anchorage	No or inadequate anchorage	Earthquake
		Frames	Steel members	Slender low damped	Earthquake; Wind
		Equipment	High-voltage units	Not qualified	Earthquake
	Transmission towers	Lattice towers	Tower	Lack of strength stiffness	Wind; Earthquake
			Foundation	Inadequate capacity	Flood; Liquefaction
			Lines	Vibration. Low damping	Wind

Table 1-2. Matrix for the various infrastructure sectors, vulnerable components and governing hazards

Subsector	Hazard	Vulnerable Component	Resilience Measure	Cost, %[2]	
				Improvement Cost	Vulnerability Reduction
Schools, Hospitals, Government buildings, Emergency centers, Hospitality and hotels	Earthquake	Wall out of plane	Add wall bracing (anchor and strongback)	15%	40%
		In plane	Add RC shear wall or Steel brace	20%	60%
		Structural irregularity	Add Shear wall, Moment frame or Braced frame	15%	40%
		Ductility	Add Column jacketing or Beam retrofit	10%	30%
		Stiffness and capacity	Add Steel brace or Steel plate shear wall	15%	50%
		Energy absorption	Add Seismic isolation or Energy dissipation damper	40%	80%
	Liquefaction	Foundation	Enlarge Spread footing or Add pile foundation	40%	80%
		Soil	Install Soil grouting or Stone/gravel columns	30%	80%
	Wind	Roof connection	Improve Roof connection anchor	10%	50%
		External curtain wall and wall	Improve Wall connection attachment	10%	30%
		Foundation connection	Add Connection anchor bolts and stiffeners	10%	50%
	Flood	Building envelope	Install flood shield	10%	80%
		Elevated construction	Elevate floodwall	15%	80%
		Foundation tie	Strengthen anchoring to foundation	10%	50%

WTP/WWTP: Plant building (Masonry/RC frame), Water storage, Equipment	Earthquake	Wall out-of-plane (Building)	Add wall bracing (anchor and strongback)	15%	40%
		In-plane (Building)	Add RC shear wall	20%	60%
		Ductility (Building)	Add column jacketing	10%	30%
		Foundation (Water storage)	Add concrete foundation	40%	50%
		Anchorage (Water storage)	Strengthen anchorage	10%	30%
		Foundation tie (Equipment)	Improve foundation tie	10%	20%
	Liquefaction	Foundation (All)	Enlarge spread footing or add pile foundation	40%	80%
		Soil (All)	Install soil grouting	30%	80%
	Wind	Roof connection (Building)	Strengthen roof connection	10%	50%
		External wall (Building)	Improve wall connection	10%	30%
		Foundation tie (Equipment)	Improve foundation tie	10%	20%
	Flood	Building envelope (Building)	Install flood shield	10%	80%

[2] Percentage of reconstruction cost (Est.)

Subsector	Hazard	Vulnerable Component	Resilience Measure	Cost, %[2]	
				Improvement Cost	Vulnerability Reduction
		Elevated construction (Building)	Construct floodwall	15%	80%
		Elevating (Equipment)	Elevate equipment	20%	80%
		Dry floodproofing (Equipment)	Install watertight barrier	15%	80%
UGP	Earthquake Liquefaction	Pipes	Replace with ductile pipes; Trenchless technologies	40%	80%
		Joints	Add flexible joints	30%	80%
Paved roads	Flood	Paved surface	Add sealant	2%	20%
			Strengthen (thicker pavement)	20%	50%
			Concrete surface	40%	80%
Bridges	Earthquake	Columns	Add fiber-reinforced polymer or steel casing	10%	60%
		Bearing	Add cable restrainers	2%	50%
		Abutment, pier seats	Add concrete bolsters	5%	50%
		Steel cross frames	Fatigue-resistant details	10%	80%
	Flood	Foundations, abutments	Add riprap	5%	20%
	Flood Liquefaction		Add new piles	40%	80%
Substation components	Earthquake	Transformer	Improved anchorage	5%	20%
		Equipment	Qualified components	10%	40%
		Equipment	Spares	10%	40%
		Support frames	Add passive dampers	5%	20%
Transmission system	Wind	Transmission tower	Strengthen steel members	20%	50%
		Transmission line	Add vibration dampers	5%	50%
	Flood, Liquefaction	Foundations	Add micro piles	40%	80%

Table 1-3. Strengthening techniques and the associated capital costs and benefit

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REFERENCED ORGANIZATIONS

AASHTO	American Association of State Highway and Transportation Officials	www.transportation.org
ACP-EU	European Union and the countries of the African, Caribbean and Pacific Group of States	https://www.gfdrr.org/en/acp-eu
ALA	American Lifelines Alliance	https://www.americanlifelinesalliance.com/
ASCE	American Society of Civil Engineers	https://www.asce.org
ASTM	American Society for Testing and Materials	http://www.astm.org/
ATC	Applied Technology Council	https://www.atcouncil.org/
BPA	Bonneville Power Administration	https://www.bpa.gov/
Caltrans	California Department of Transportation	https://dot.ca.gov/
CCRIF	Caribbean Catastrophe Risk Insurance Facility Segregated Portfolio Company	https://www.ccrif.org/
CDMP	Caribbean Disaster Mitigation Project	https://www.oas.org/cdmp/
CEHI	Caribbean Environmental Health Institute	https://knowledge.unccd.int/kss/caribbean-environmental-health-institute-cehi
CHARIM	Caribbean Handbook on Risk Information Management	http://www.charim.net/
CIMH	Caribbean Institute for Meteorology and Hydrology	www.cimh.edu.bb
EPRI	Electric Power Research Institute, Inc	www.epri.com
FDOT	Florida Department of Transportation	https://www.fdot.gov/
FEMA	Federal Emergency Management Agency	https://www.fema.gov/
FHWA	Federal Highway Administration	https://www.fhwa.dot.gov
FIMA	Federal Insurance and Mitigation Administration	https://www.fema.gov/about/offices/insurance-mitigation
HA	Hong Kong Housing Authority	https://www.housingauthority.gov.hk/en/
ICC	International Code Council	https://www.iccsafe.org/
IDB	Inter-American Development Bank	https://www.iadb.org/en
IEEE	Institute of Electrical and Electronics Engineers	https://www.ieee.org/
INAPA	Instituto Nacional de Aguas Potables y Alcantarillados	http://www.inapa.gob.do/
IWR	Institute for Water Resources	https://www.iwr.usace.army.mil/
MISO	Midcontinent Independent System Operator	https://www.misoenergy.org/
MWH	MWH Global Inc.	http://www.mwhglobal.com/
NIBS	National Institute of Building Sciences	https://www.nibs.org
NIST	National Institute of Standards and Technology	https://www.nist.gov/
NOAA	National Oceanic and Atmospheric Administration	https://www.noaa.gov/
OAS	Organization of American States	http://www.oas.org/
ODPM	Office of Disaster Preparedness and Management	http://www.odpm.gov.tt/
OECS	Organisation of Eastern Caribbean States	http://www.oecs.org/
OSSPA	Oregon Seismic Safety Policy Advisory	https://www.oregon.gov/gov/policy/orr/pages/index.aspx
PAHO	Pan American Health Organization	http://www.paho.org/
USACE	The U.S. Army Corps of Engineers	https://www.usace.army.mil
USAID	United States Agency for International Development	https://www.usaid.gov/
USGS	The United States Geological Survey	https://www.usgs.gov
WBG	The World Bank Group	https://www.worldbank.org/
WHO	World Health Organization	https://www.who.int/

ACRONYMS, ABBREVIATIONS, AND NOTATIONS

AAL	Average annual loss
AC	Asbestos cement (concrete)
ACSS	Aluminum Conductor Steel Supported
ADT	Average daily traffic
ATPB	Asphalt Treated Permeable Base
BRB	Buckling restrained braces
CBR	California Bearing Ratio
CGMC	Construction Manager General Contractor
CI	Cast iron
CUBiC	Caribbean Uniform Building Code
DBB	Design bid build
DBE	Design Basis Earthquake
DI	Ductile iron
DS	Damage state
FRP	Fiber-Reinforced Polymer
GS	Galvanized steel
HDPE	High-Density Polyethylene
HMA	Hot mix asphalt
IBC	International Building Code
IRI	International Roughness Index
LCCA	Life-cycle cost analysis
MGD	Millions of gallons per day
MPa	Mega Pascal
MTEP	MISO Transmission Expansion Plan
MVA	Megavolt-ampere
OOP	Out of plane
P3	Public-private partnership
PASS	Performance Assessment Scoring System
PC	Precast (prestressed) concrete
PCC	Portland Concrete Cement
PGA	Peak ground acceleration
PGD	Permanent ground displacement
PGV	Peak ground velocity
PGWS	Peak gust wind speed
Psi	Pound per square inch
PVC	Polyvinyl chloride
RC	Reinforced concrete
RC/CF	Reinforced/concrete frame with infill wall
S_a	Spectral acceleration
S_d	Spectral displacement
TMD	Tuned mass damper
UGP	Underground pipe
URM	Unreinforced masonry
WS	Welded steel
WTP	Water treatment plant
WWTP	Wastewater treatment plant

1. INTRODUCTION

1.1 Overview

Infrastructure comprises a critical component of community health and economic growth. In recent natural disasters, including earthquakes, hurricanes, and floods, significant damage to infrastructure occurred, resulting in economic losses and delayed recovery.

The Caribbean region is undergoing growth and the tourist industry is one of the main engines of economic growth for these countries. The area is also vulnerable to natural hazards. Past earthquakes have had decades-long impact on countries within the region, and frequent hurricanes result in significant structural damage on an annual basis.

To address such vulnerabilities and to improve the resilience of Caribbean communities, this project was undertaken to assess the current state of selected infrastructure and to provide recommendations for improvements in the subject counties listed in Table 1-1, with the objective of evaluating the costs and benefits associated with strengthening options.

No	Country	Area, km^2	Population, millions	GDP, US$ billion
1	Bahamas	13,900	0.39	12.6
2	Barbados	439	0.28	5.4
3	Belize	23,000	0.41	3.5
4	Dominican Republic	48,670	10.7	216
5	Guyana	215,000	0.79	13.5
6	Haiti	27,800	11.1	20
7	Jamaica	10,900	2.9	27
8	Antigua and Barbuda	440	0.1	2.7
9	Dominica	750	0.07	0.7
10	Grenada	349	0.11	1.8
11	Saint Kitts and Nevis	261	0.05	1.8
12	Saint Lucia	617	0.18	2.7
13	Saint Vincent and the Grenadines	389	0.11	1.4
14	Sint Maarten	34	0.04	0.4
15	Suriname	163,800	0.58	9
16	Trinidad and Tobago	5,130	1.36	45.1

Table 1-1. General information for subject countries (various sources)

1.2 Scope and objectives

The objective of the current project is to undertake a high-level conceptual, but detailed, analysis of engineering options for increased infrastructure resiliency in the Caribbean region, as well as an estimate of the cost of these options. The following are considered:

- Infrastructure considered:
    - Buildings (focusing on critical public buildings, such as schools, hospitals, government buildings, emergency centers, and hospitality/hotels)
    - Water (water and wastewater treatment plants, and pipelines)
    - Transport (roadway bridges and primary paved roads)
    - Power (power plants, substations, and distribution systems)

- Natural hazards considered:
    - Floods
    - Hurricanes (wind)
    - Earthquakes (both shaking and liquefaction)

Throughout the remainder of this report, the natural hazards above may be defined as follows:

- Floods: When specifying this hazard, may be noted as "flood" or "flooding", and including river, storm surge, etc.
- Earthquakes: When specifying this hazard, may be noted as "ground shaking" and/or "liquefaction." For ground shaking, may be noted as earthquake/earthquake ground shaking/ground shaking/shaking. For liquefaction, may be noted as earthquake liquefaction/liquefaction. As much as possible, ground shaking and liquefaction will be distinguished.
- Hurricanes: When specifying this hazard, may be noted as "wind."

1.3 Limitations

As part of this study, significant outreach in the Caribbean region was undertaken to collect specific data relevant to the subject countries. When available, this data is used in the report. When such data was not readily available, engineering judgement was used and augmented by the knowledge of construction and design practices in the Caribbean.

It is further assumed that new construction and most infrastructure constructed after the year 2000 would have been designed per provisions of modern codes, and thus, are likely more robust than earlier construction. Accordingly, new construction is not considered in this report, and the recommendations are designed for retrofitting of existing infrastructure.

The focus of the work will be on structural aspects of resilient development. As such, physical damage and the costs associated with such damage will be solely considered. Other factors, such as casualties, repair time, and secondary or user costs, are not included.

1.4 Subject countries and vulnerable infrastructure

As noted earlier, there are 16 subject countries, several hazards, and a number of infrastructure sectors that need to be considered. To streamline the process, the 16 countries were placed in four groups; see Table 1-2 and Figure 1-1. The grouping is based on factors like geographical location, governing natural hazards, and relative size or population of the countries. For each group, the critical natural hazards were identified, and subsectors specified, as seen in Table 1-2.

<table>
<tr><th>Group</th><th>Country</th><th>Hazard</th><th>Buildings</th><th>Water</th><th>Transport</th><th>Power</th></tr>
<tr><td rowspan="3">1</td><td>Suriname</td><td rowspan="3">Earthquake</td><td rowspan="3">Government, Emergency, Schools, Hospitals, Hotels</td><td rowspan="10">Water treatment plants (WTP)

Wastewater treatment plants (WWTP)

Underground pipes (UGP)</td><td rowspan="3">Bridges</td><td rowspan="3">Substations</td></tr>
<tr><td>Trinidad and Tobago</td></tr>
<tr><td>Guyana</td></tr>
<tr><td rowspan="3">2</td><td>Belize</td><td rowspan="3">Earthquake
Wind</td><td rowspan="3">Government, Emergency, Schools, Hospitals, Hotels</td><td rowspan="3">Bridges</td><td rowspan="3">Substations</td></tr>
<tr><td>Haiti</td></tr>
<tr><td>Dominican Republic</td></tr>
<tr><td rowspan="4">3</td><td>Antigua and Barbuda</td><td rowspan="4">Wind
Flood</td><td rowspan="4">Schools, Hospitals, Hotels</td><td rowspan="4">Roads</td><td rowspan="4">Transmission lines</td></tr>
<tr><td>Dominica</td></tr>
<tr><td>Grenada</td></tr>
<tr><td>Saint Kitts and Nevis</td></tr>
</table>

	Saint Lucia					
	Saint Vincent and the Grenadines					
	Sint Maarten					
	Barbados					
4	Jamaica	Wind Flood	Schools, Hospitals, Hotels		Roads	Transmission lines
	Bahamas					

Table 1-2. Vulnerable infrastructure and associated natural hazards in subject countries (WB, 2020)

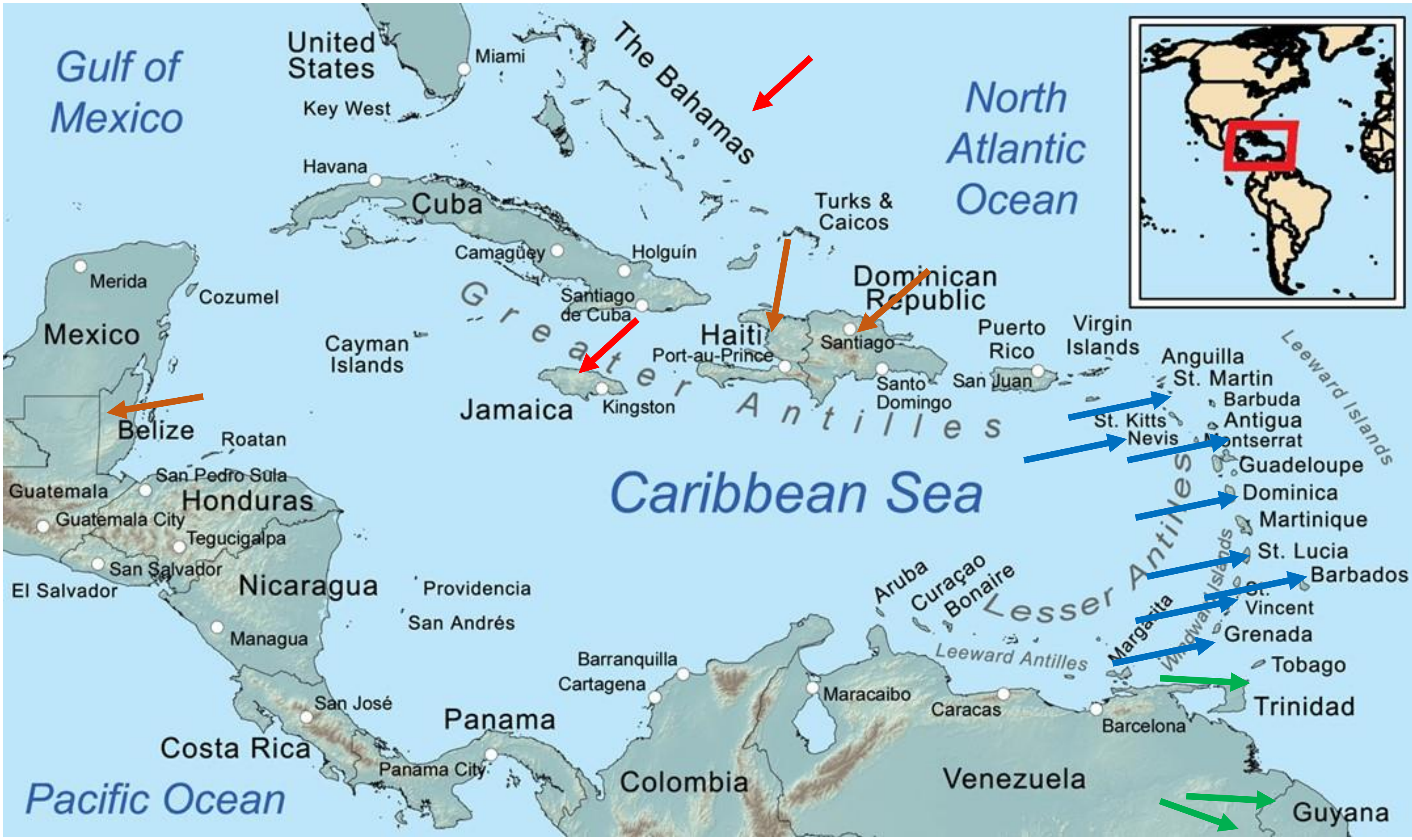


Figure 1-1. Grouping of countries (Kmusser, 2020)

LEGEND

→ = Group 1 → = Group 2 → = Group 3 → = Group 4

1.5 Evaluation methodology

For each class of substructure, the sector infrastructure types most vulnerable to the critical natural hazards in that region were identified. Since the infrastructure design and construction practice are similar in the subject countries, representative states were used as examples. However, the discussion extends to other countries, and when available, data from other countries are presented. For example, for countries in Groups 1 and 2, the emphasis for transportation infrastructure will be on elevated bridges and for Groups 3 and 4, on roads. The hazard and infrastructure data in Table 1-2 are consistent with experience from previous events, which has shown that bridges are vulnerable to earthquakes (ground shaking and liquefaction), whereas flooding—including floods as a result of hurricane storm surge—severely impacts roadways. The assessment is based on structural types and components (not hazard-, country-, or group-based) as discussed in subsequent chapters of this report.

1.6 Organization of the report

Chapter 1 includes an introduction to the study. Chapter 2 provides information on natural hazards. Chapters 3, 4, 5, and 6 discuss critical buildings, water, transport, and power sectors, respectively. In these chapters, general typology, code provisions, vulnerable components, strengthening techniques, and the associated costs are discussed. Chapter 7 discusses additional measures that can be utilized for improved resiliency, including improved quality of construction and regular preventive maintenance. Chapter 8 presents a brief discussion on cost and financing and life-cycle cost analysis. Conclusions and recommendations are presented in Chapter 9 and a list of references in the final chapter. APPENDIX A summarizes a list of key reference files used in this report.

2. NATURAL HAZARDS

2.1 Introduction

In this chapter, the key natural hazards impacting the subject countries are discussed. Given that the countries are either island nations or continental states with large coastlines, and located in hurricane-prone zones, hurricanes and resulting flooding are the most common natural hazards. Table 2-1 (ThinkHazard, 2020) presents the relative importance of various hazards for the subject countries. Although earthquakes are less frequent, on an annualized basis, they can be more consequential. As such, earthquakes (both ground shaking and liquefaction) are also considered. Earthquakes impact a number of countries in the Caribbean; see Table 2-2 (CHARIM, 2020).

Country	Earthquake	Hurricane	Flood (Coastal and river)
Suriname	Low	Low	High
Trinidad and Tobago	Medium	High	High
Guyana	Low	Low	High
Belize	Medium	High	High
Haiti	Medium	High	High
Dominican Republic	Medium	High	High
Antigua and Barbuda	Medium	No data	Medium
Dominica	Medium	No data	High
Grenada	Medium	No data	Medium
Saint Kitts and Nevis	Medium	No data	Medium
Saint Lucia	Medium	No data	Medium
Saint Vincent and the Grenadines	Medium	No data	Medium
Sint Maarten	No data	No data	No data
Barbados	Medium	No data	Medium
Jamaica	Medium	High	High
Bahamas	Medium	High	High

Table 2-1. Relative importance of various hazards in different countries (ThinkHazard, 2020)

Country	Earthquake	Wind	Storm Surge	Flood
Belize	3.2%	30%	16.4%	50.4%
Dominica	19.1%	34.8%	46.2%	--
Grenada	29%	34.1%	36.9%	--
Saint Lucia	10.8%	43.7%	45.5%	--
Saint Vincent and the Grenadines	11.4%	26.9%	61.7%	--

Table 2-2. Hazard contribution to AAL[3] for selected countries (adapted from CHARIM, 2020)

2.2 Natural Hazards

2.2.1 Earthquake hazard

2.2.1.1 Ground shaking

Figure 2-1 presents the seismic hazard map for selected subject countries (CDMP, 2020). The data is shown for peak ground acceleration (PGA) of a 475-year (20% probability of occurrence every 50 years) earthquake. This level of seismicity is typically used in construction of new infrastructure and is expected to provide life preservation (ordinary structures) or continuous operation (critical infrastructure). The

[3] Average Annual Loss

acceleration contours presented in the figure correspond to bedrock motions and need to be adjusted for the site to account for soil amplification. The PGA values typically vary from 0.1 to 0.4 g in the figure, corresponding to low-to-moderate seismicity.

2.2.1.2 Liquefaction

Figure 2-2 (Kraft, 2013) presents the liquefaction susceptibility map for Jamaica and Trinidad. Because many of the subject countries have low-lying areas with high ground water elevation and sandy soil, the coastal areas could experience liquefaction in large earthquakes. However, this is considered a secondary hazard compared to ground shaking.

2.2.2 Wind hazard

The Pan American Health Organization (PAHO) presents the developed wind speed maps (2019) for the Caribbean region for 50- to 1700-year events; see Figure 2-3. Table 2-3 lists the wind speed associated with various categories of hurricanes. Table 2-4 presents the wind speed for various subject countries depending on different return intervals. The listed wind speeds can be used with American Society of Civil Engineers (ASCE) procedures for the design of new structures.

Category	Wind speed, miles/hour (mph)
1	74-95
2	96-110
3	111-129
4	130-156
5	157+

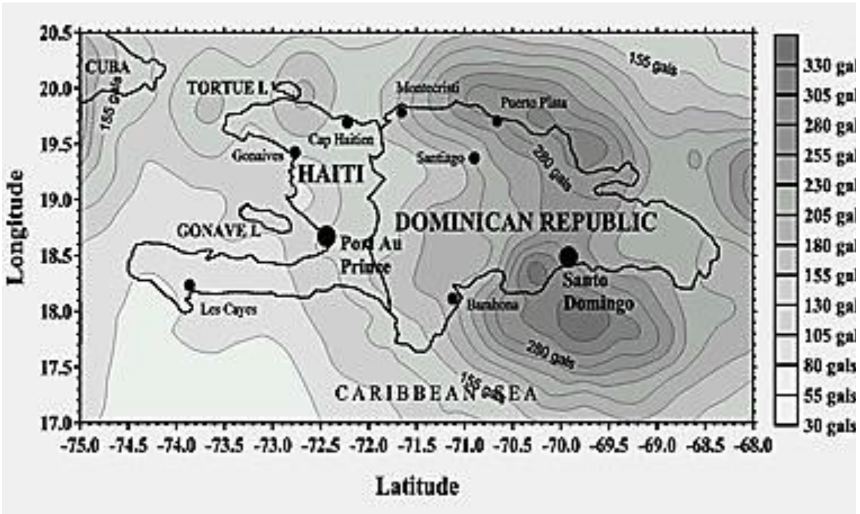
Table 2-3. Hurricane category and sustained wind speed (National Hurricane Center, 2020)

	Return period, years			
Country	**50**	**100**	**700**	**1700**
Suriname	--	--	--	--
Trinidad and Tobago	61	85	136	156
Guyana	--	--	--	--
Belize	106	128	165	177
Haiti	--	--	175	185
Dominican Republic	--	-	170	185
Antigua and Barbuda	121	134	160	168
Dominica	106	124	159	171
Grenada	85	107	165	168
Saint Kitts and Nevis	125	138	163	170
Saint Lucia	101	119	155	172
Saint Vincent and the Grenadines	93	111	155	171
Sint Maarten	129	141	168	178
Barbados	92	112	152	169
Jamaica	--	--	150	170
Bahamas	121	132	163	180

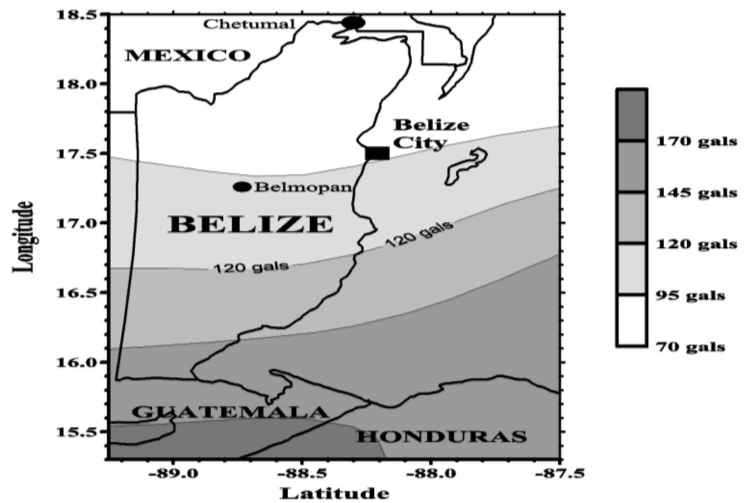
Table 2-4. Peak gust wind (mph) for subject countries (adapted from PAHO, 2019)

2.2.3 Flood hazard

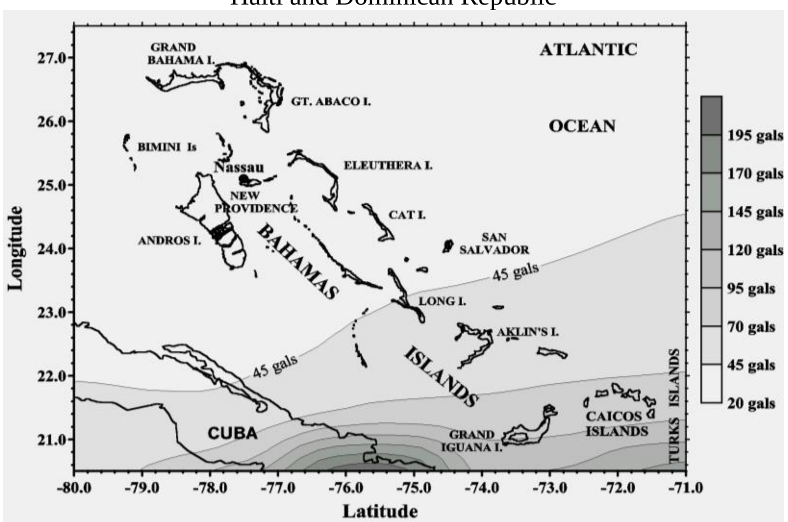
Figure 2-4 presents the flood hazard maps for some of the Caribbean countries. The countries, closely located to the sea and with many rivers, are susceptible to storm surge, flash, and river flooding. Additionally, if proper drainage is not provided, urban flooding could occur. In this report, all sources of flooding are treated as having a similar impact.



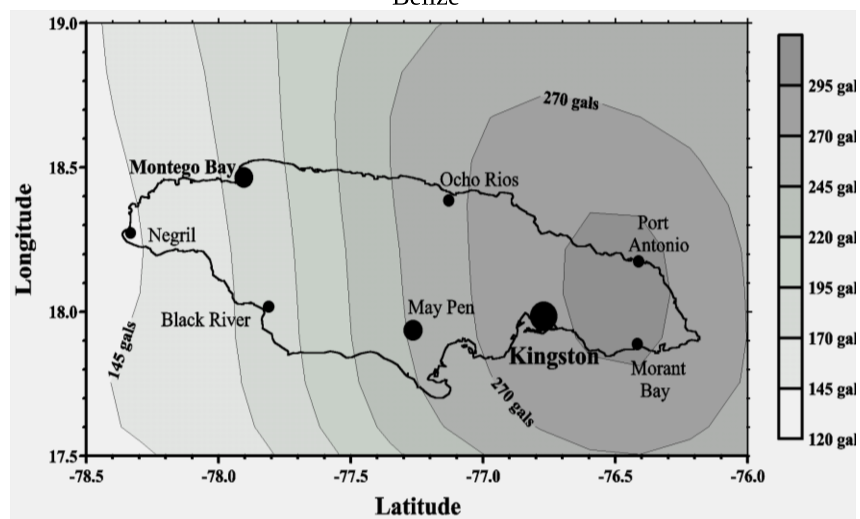
Haiti and Dominican Republic



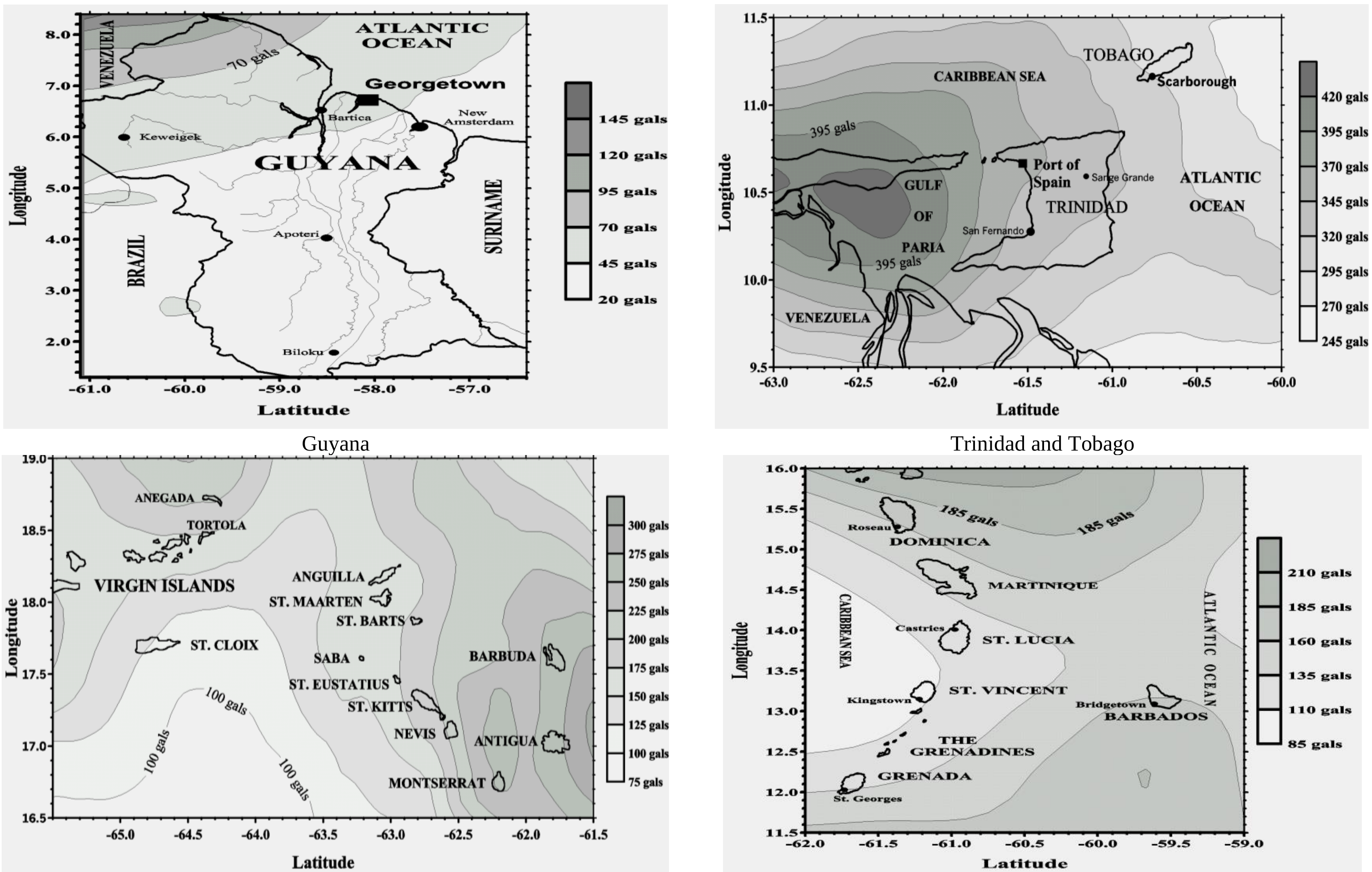
Belize



Bahamas



Jamaica



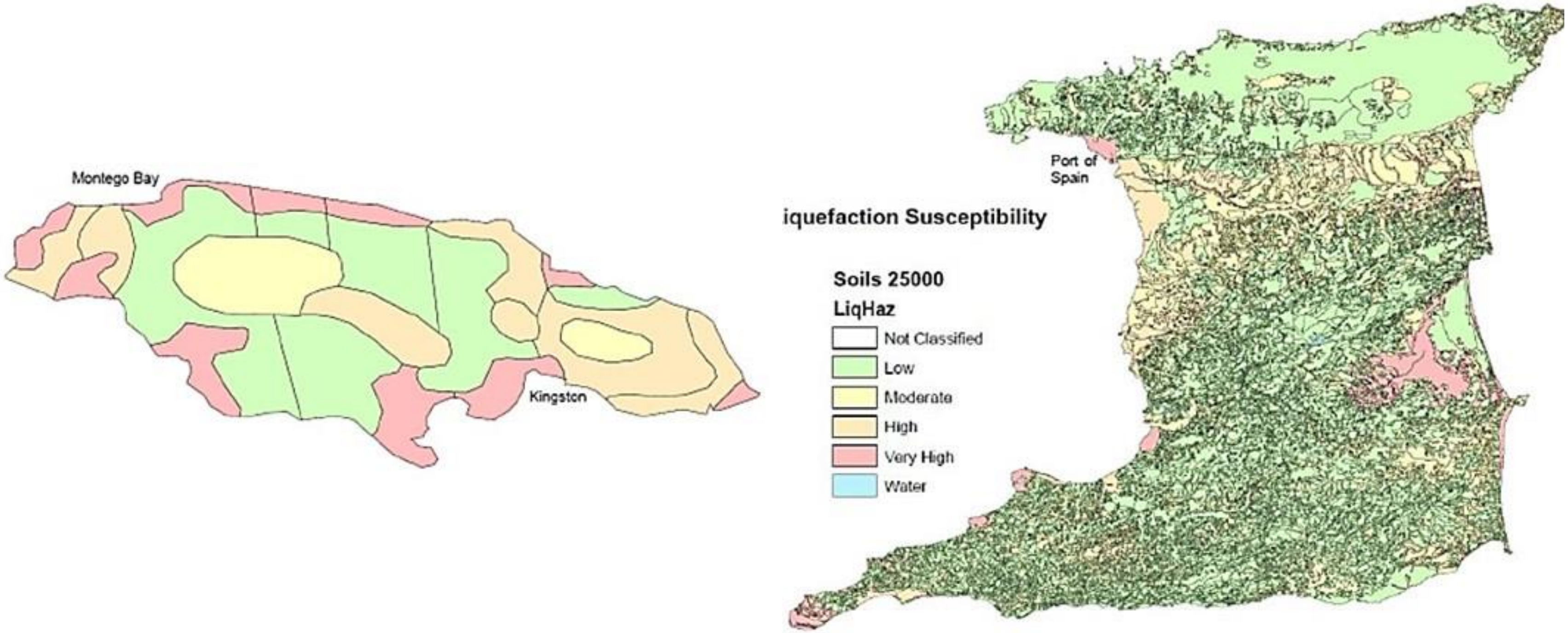
Guyana

Trinidad and Tobago

Leeward Islands

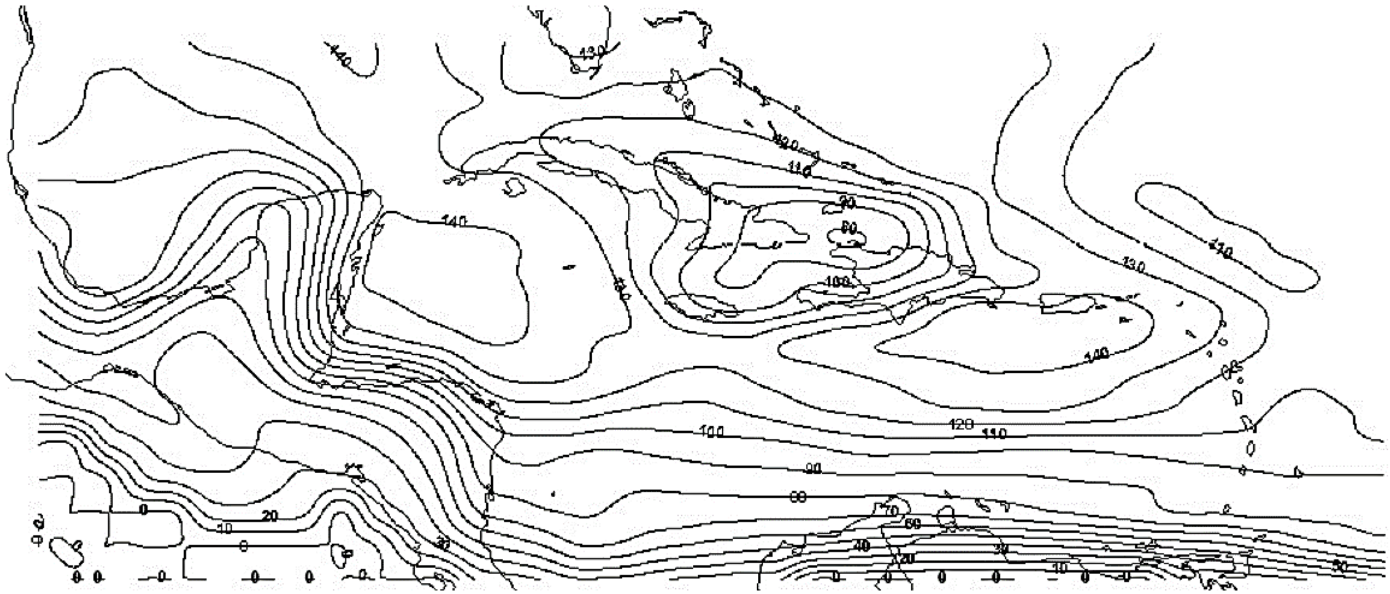
Windward Islands

Figure 2-1. Seismic hazard maps (CDMP, 2020)

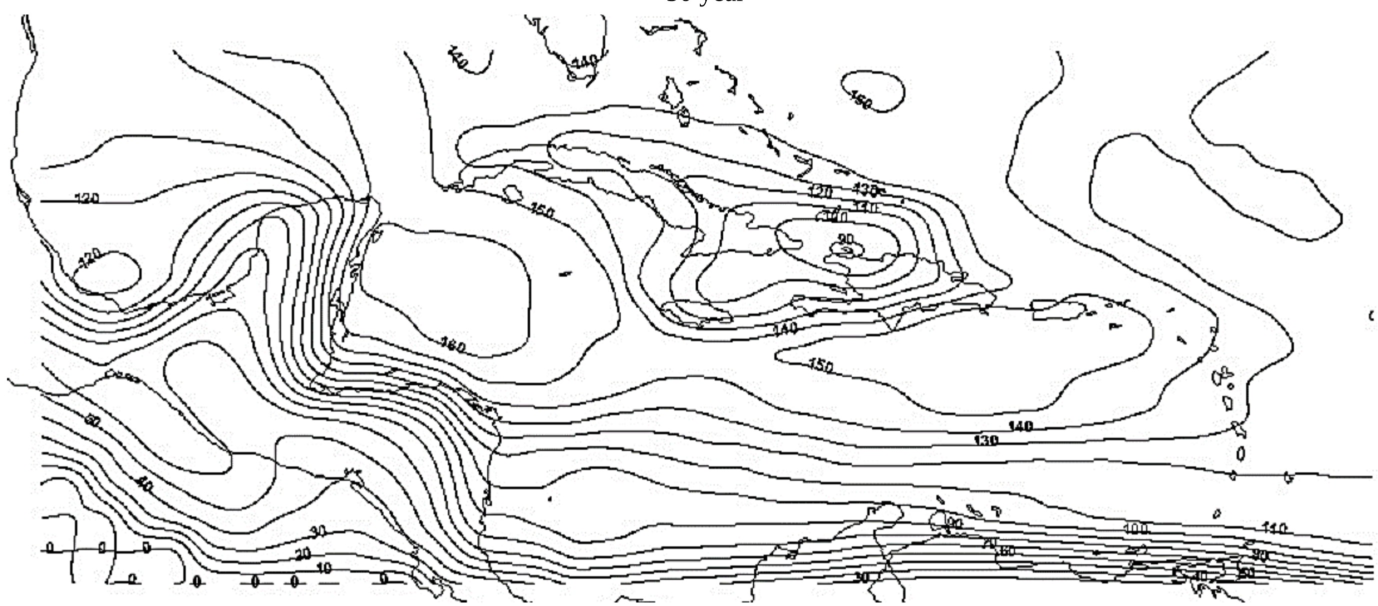


Jamaica Trinidad

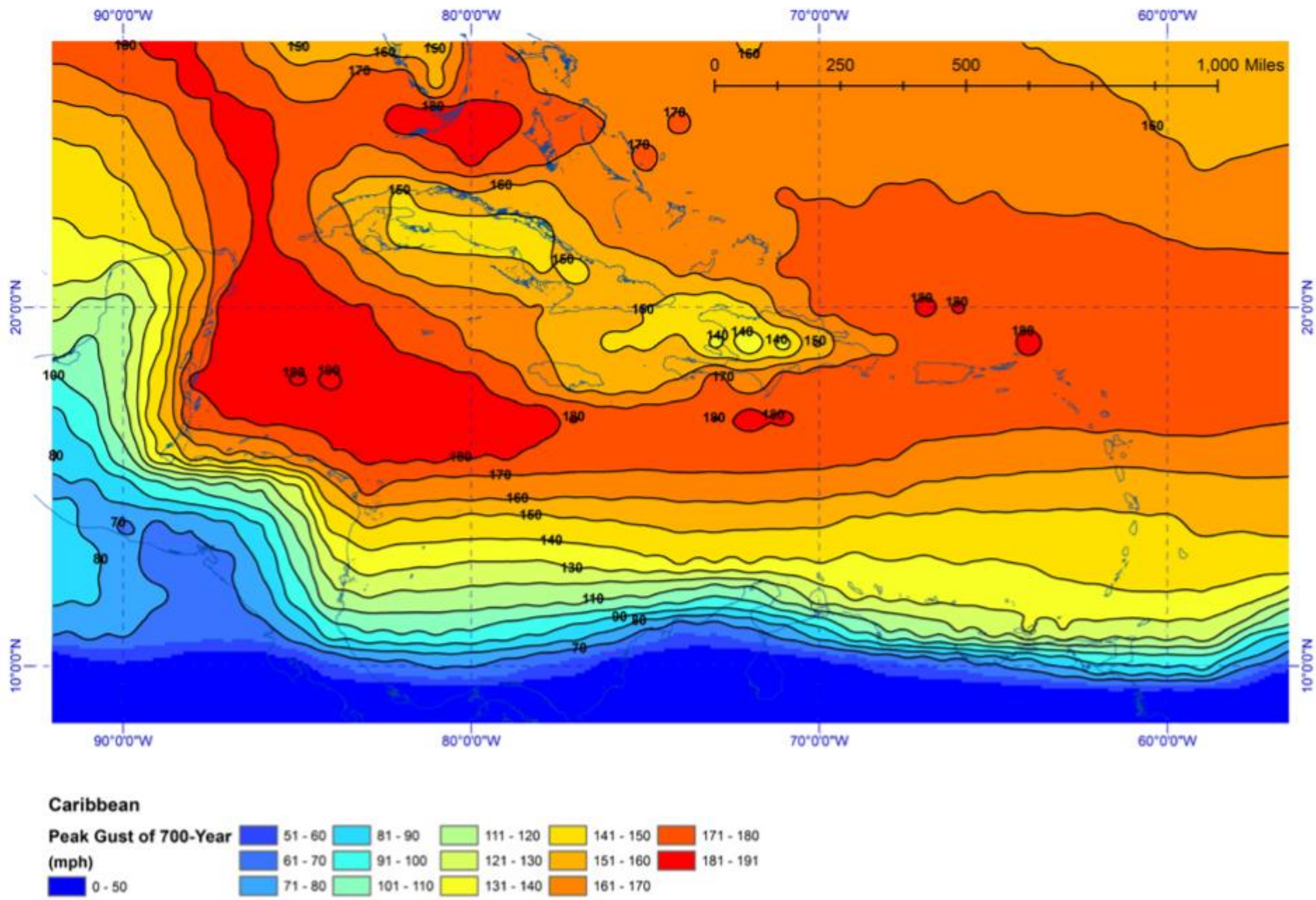
Figure 2-2. Liquefaction hazard maps (Kraft, 2013)



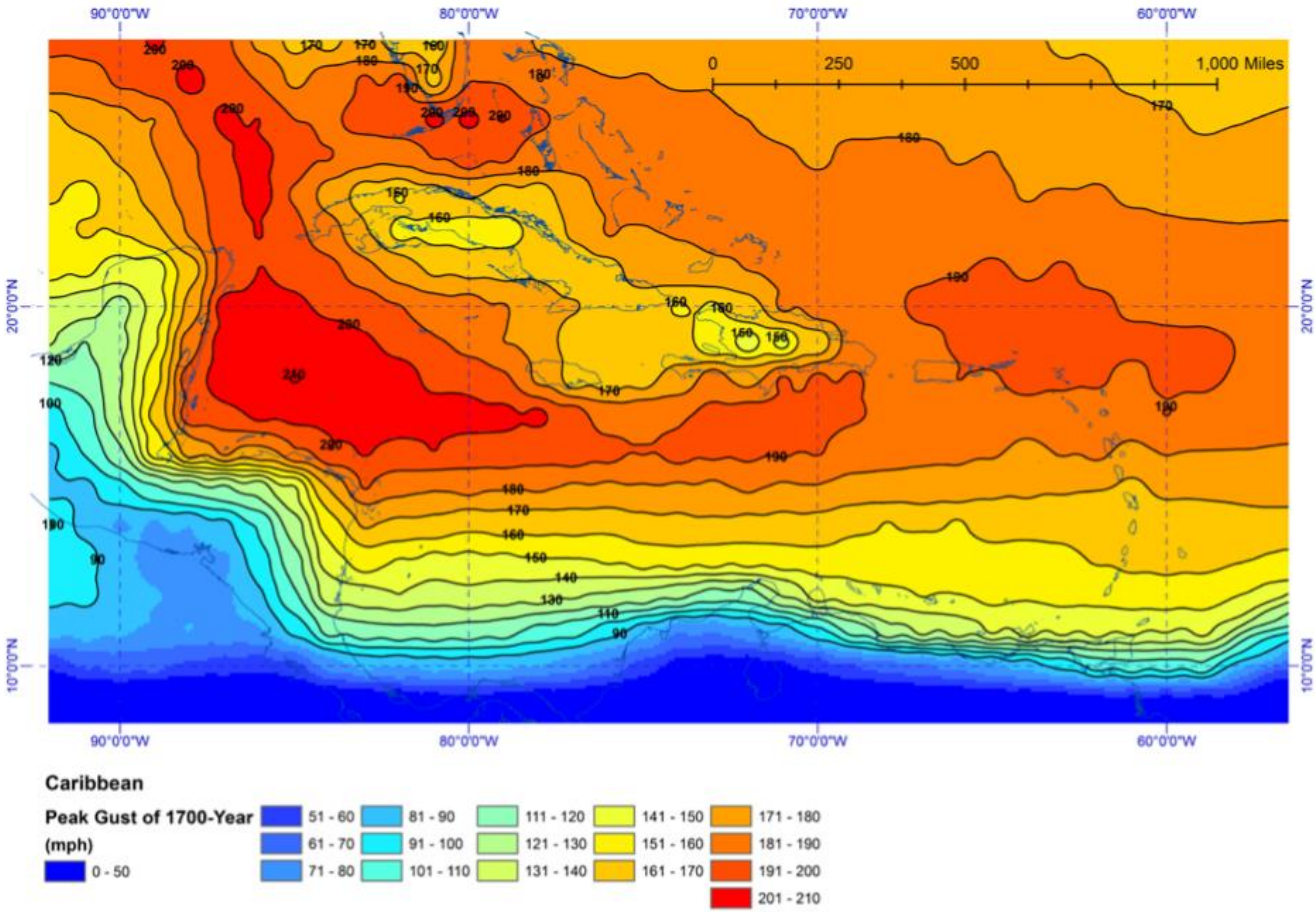
50 year



100 year

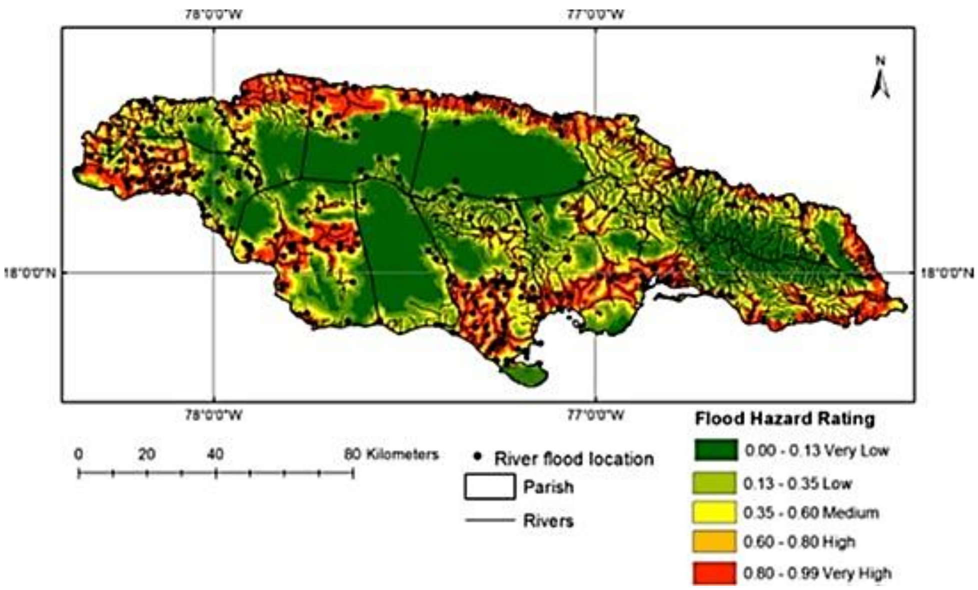

90°0'0"W
80°0'0"W
70°0'0"W
60°0'0"W
20°0'0"N
10°0'0"N
0
250
500
1,000 Miles
Caribbean
Peak Gust of 700-Year (mph)
0 - 50
51 - 60
61 - 70
71 - 80
81 - 90
91 - 100
101 - 110
111 - 120
121 - 130
131 - 140
141 - 150
151 - 160
161 - 170
171 - 180
181 - 191

700 year

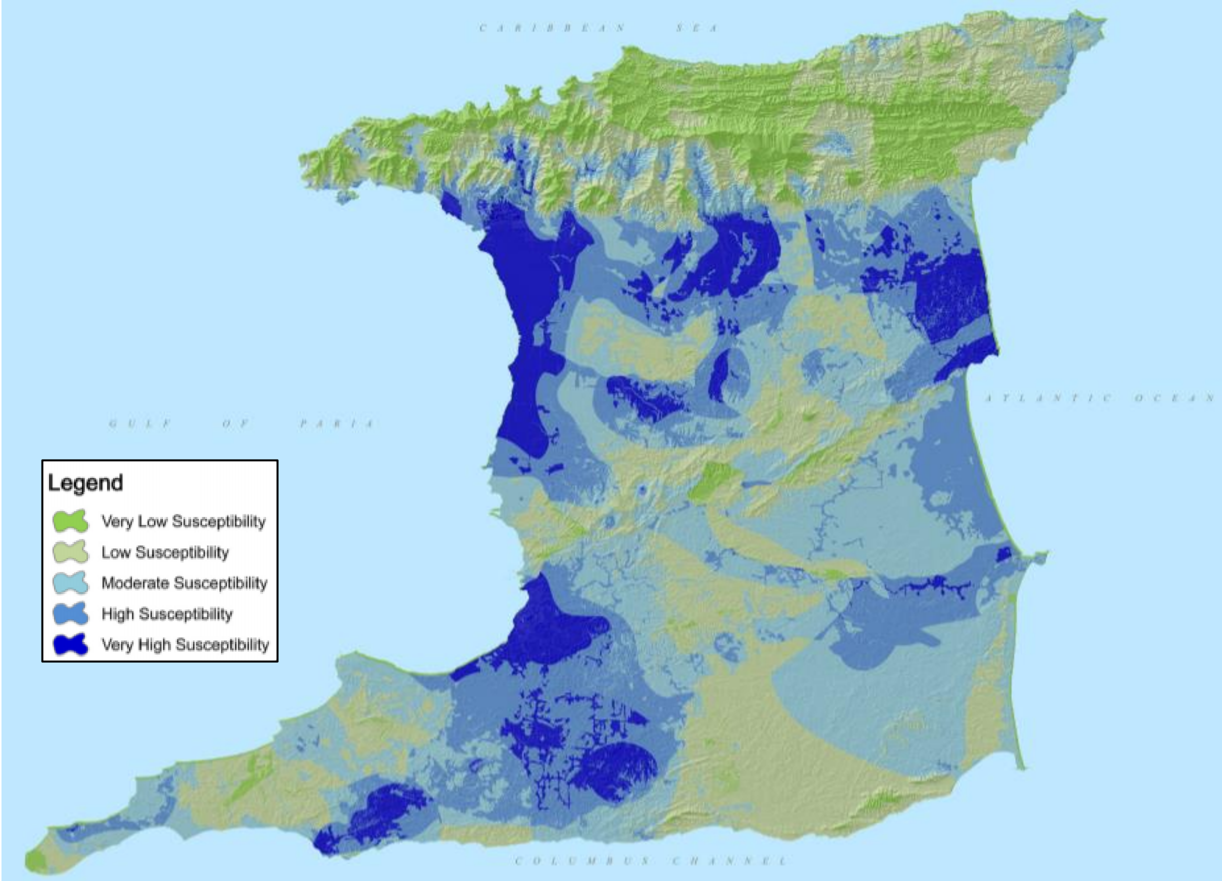


1700 year

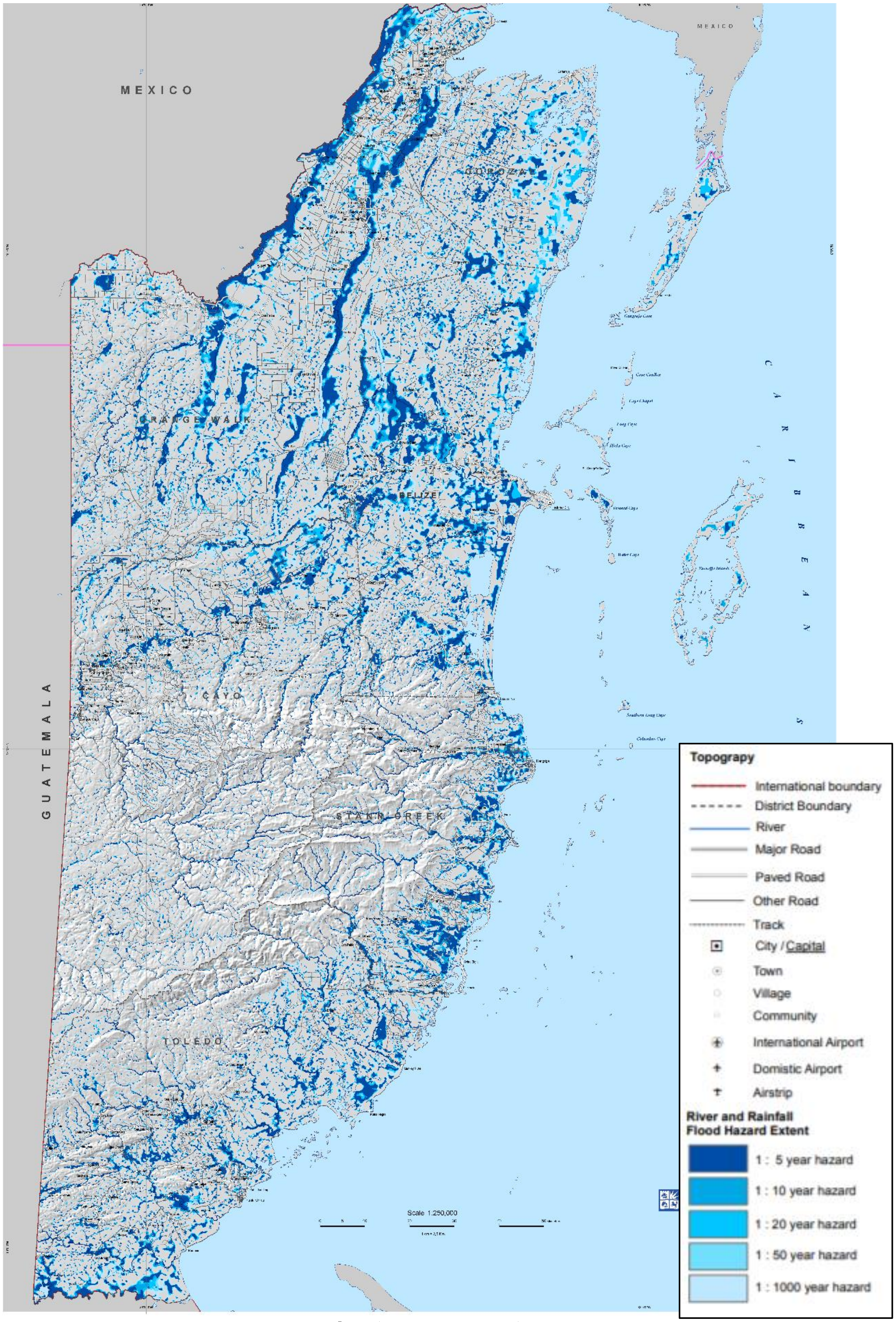
Figure 2-3. Wind hazard maps (PAHO, 2019)



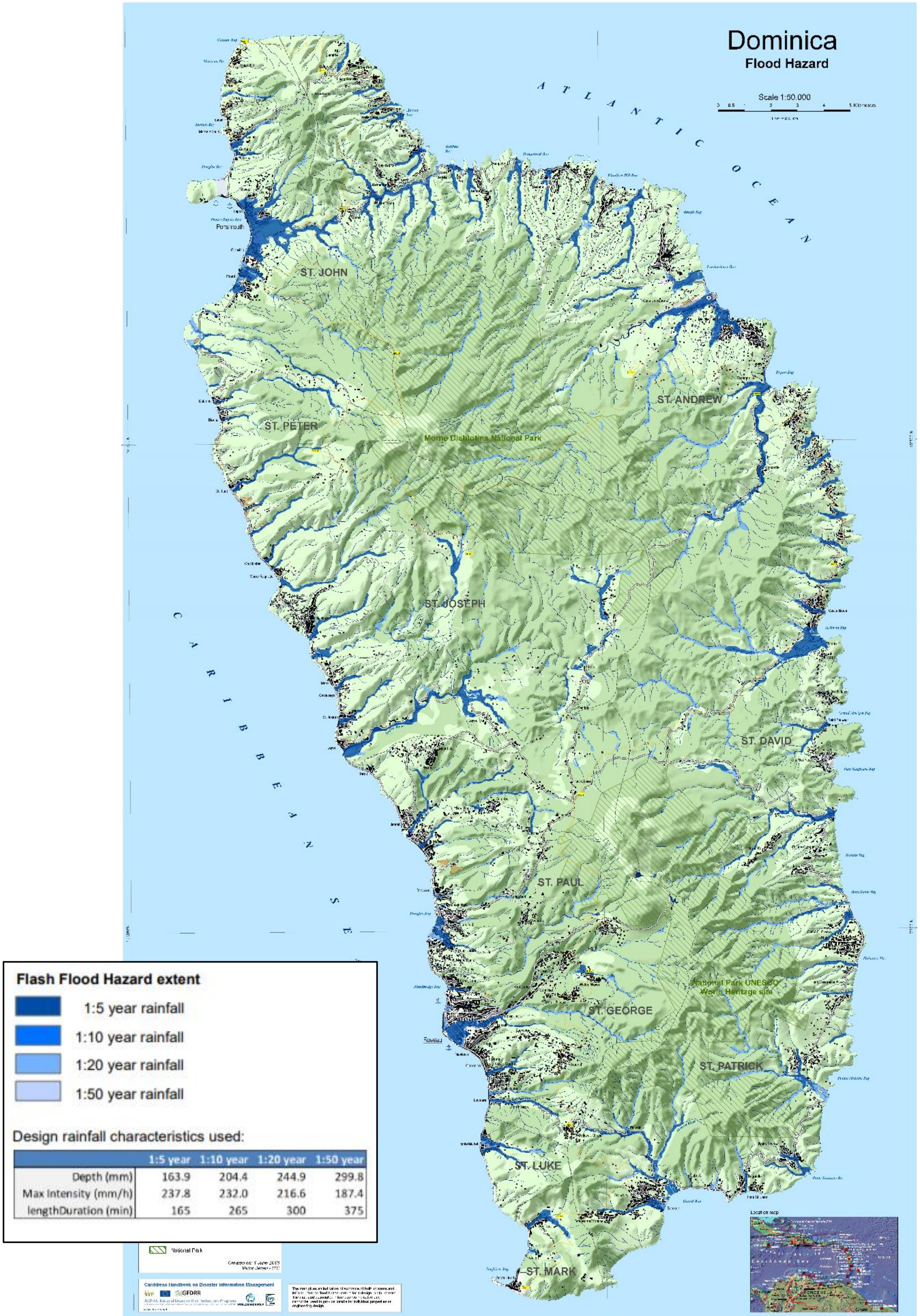
Jamaica (Nandi et al., 2016)



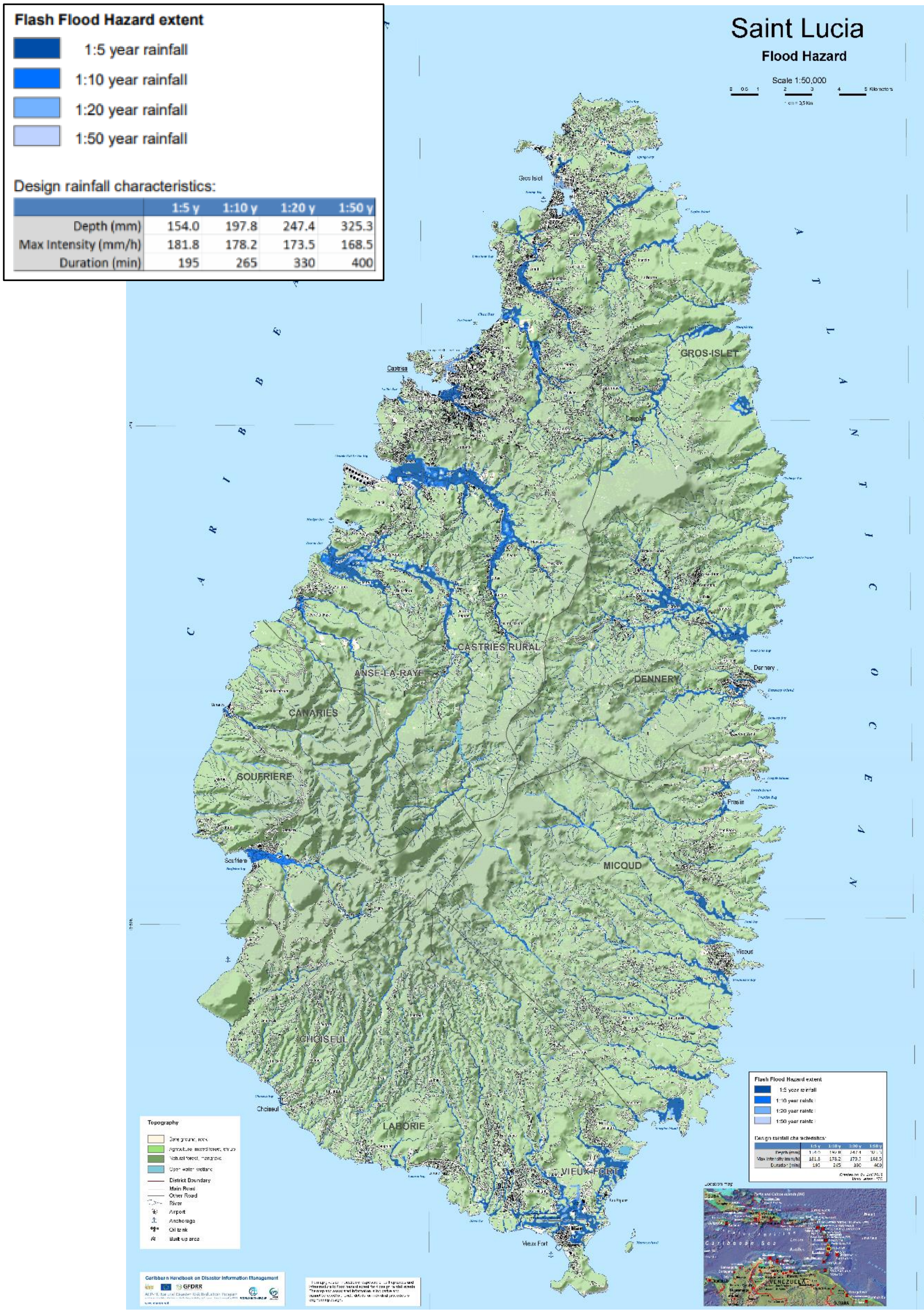
Trinidad and Tobago (ODPM, 2020)



Belize (CHARIM, 2020)



Dominica (flashflood) (CHARIM, 2020)



Saint Lucia (flashflood) (CHARIM, 2020)

Figure 2-4. Flood hazard maps

2.3 Infrastructure impact

In the 2020 Atlantic hurricane season alone, there were a number of hurricanes and tropical storms that caused significant damage in the Caribbean. See Figure 2-5 for an example.

- Hurricane Isaias impacted the Leeward Islands, Haiti, the Dominican Republic, and Bahamas. In the Bahamas, electricity was lost and there was damage to houses and roads.
- Hurricane Laura had an impact on the Dominican Republic, Haiti, and Jamaica. Schools were closed in Antigua.
- Hurricane Marco caused significant flooding in the Caribbean, including in Haiti and the Dominican Republic.
- Hurricane Nana caused significant flooding in the Caribbean, including in Belize.

Inundated road (Bahamas: Hurricane Isaias)

Damaged buildings (Haiti: Hurricane Laura)

Flooding (Dominican Republic: Hurricane Marco)

Flooded bridge (Belize: Hurricane Nana)

Figure 2-5. Damage to infrastructure in the Caribbean, 2020 Hurricane season (various sources)

For the various infrastructure types considered in this group, some of the critical hazards are considered less "critical" and thus are de-emphasized in this report. Shaded entries in the infrastructure-hazard matrix of Table 2-5 are the hazards that are considered most critical.

Sector	Subsector	Earthquake[4]	Wind	Flood
Buildings	Public buildings			
Water	Water treatment plant			
	Wastewater treatment plant			
	Underground pipes			

[4] Ground shaking and/or liquefaction

Transport	Paved Roads			
	Bridges			
Power	Power plants			
	Substations			
	Transmission towers and lines			

Table 2-5. Matrix of considered hazards for various infrastructure

2.4 Selection of governing hazard

In this report, a number of hazards and infrastructure types are considered for the 16 subject countries. To streamline the assessment process, for each infrastructure subsector, several structural (construction) types were selected based on the expected typologies in the subject countries. Next, for each structural type, several components to consider along with the governing hazards were chosen; see Table 2-6. These components, their vulnerabilities, and the steps to improve resiliency are discussed in subsequent chapters.

Sector	Subsector	Type[5]	Components to Consider	Governing Hazard(s)
BUILDINGS	Schools, Hospitals. Government buildings, Emergency centers, Hospitality and Hotels	URM	Wall	Earthquake
			Roof	Wind
		RC/CF with infill wall	Shape and figure	Earthquake
			Wall	Earthquake
			Roof	Wind
			Opening	Flood
		RC frame	Shape and figure	Earthquake
			Ductility	Earthquake
			External wall	Wind
		Steel frame	Stiffness	Earthquake, Wind
			Strength	Earthquake
			Roof	Wind
		Steel frame high-rise	Stiffness	Earthquake, Wind
			Ductility	Earthquake
			Cladding	Wind
		Wood frame	Roof	Wind
			Foundation	Wind, Flood
			Elevation	Flood
		Foundation		Liquefaction
WATER	Water treatment plants Wastewater treatment plants	Plant building (Masonry/RC frame)	Wall	Earthquake
			Ductility	Earthquake
			Roof	Wind
			External wall	Wind
			Opening	Flood
			Elevation	Flood
		Water storage	Foundation	Earthquake
			Anchorage	Earthquake
		Equipment (mechanical and electrical)	Foundation	Earthquake, Wind
			Elevation	Flood
			Floodproofing	Flood
		Foundation		Liquefaction

[5] RC = Reinforced concrete; URM = Unreinforced masonry; RC/CF = Reinforced Concrete/Concrete Frame

Sector	Subsector	Type[5]	Components to Consider	Governing Hazard(s)
	Underground pipes	Pipe	Brittle pipes	Earthquake, liquefaction, flood
		Joints	Conventional	Earthquake, liquefaction
TRANSPORT	Paved roads	Asphalt paved	Subsurface	Flood
			Surface	Flood
	Bridges	Concrete girder	Columns	Earthquake
			Joints	Earthquake
		Steel truss	Members	Earthquake
			Connections	Earthquake
		Steel girder	Bearings	Earthquake
			Cross frames	Earthquake
		Steel bridges	Welded connections	Wind
		Suspension	Bridge stability	Wind
		Foundation		Earthquake, Flood
POWER	Power plants	Low-rise RC	Design	Earthquake
			Grade construction	Flood
			Nonstructural components	Earthquake
			Foundation	Earthquake, liquefaction
	Substations	URM Control buildings	Design construction	Earthquake
			Grade construction	Flood
			Battery racks, cable trays, etc.	Earthquake
			Foundation	Earthquake, liquefaction
		Transformers	Anchorage	Earthquake
		Frames	Steel members	Earthquake, wind
		Equipment	High-voltage units	Earthquake
	Transmission & Distribution lines	Lattice towers	Tower	Wind, Earthquake
			Foundation	Flood, Liquefaction
			Lines	Wind

Table 2-6. Matrix of considered typologies for various infrastructure

2.5 Discussion

The selected countries in the Caribbean are susceptible to earthquakes, windstorms, and flooding. Many of the countries experience frequent hurricane and storm surge impacts. Although less frequent, earthquakes have devastated a number of the countries in the region with long-term adverse effects. In this report, classes and subclasses of critical infrastructure will be studied. The focus will be on the select structural types that are more typical in the region, and components of these structural types that are the most vulnerable.

3. BUILDING INFRASTRUCTURE

3.1 Introduction

For building infrastructure in the Caribbean, the typical structural types, governing hazards, and vulnerable components are summarized in Table 3-1. Based on damages to buildings seen in past natural disasters, heavier buildings are generally more at risk in earthquakes and lighter buildings are usually more vulnerable to hurricane winds and flood events. The vulnerable components to consider are then identified for each governing hazard according to the representative structural types, which are discussed in the following sections in further detail. Since the structural type is the main factor to scale the disaster vulnerability of a building, the discussions, especially on strengthening techniques, focus on the structural components of buildings.

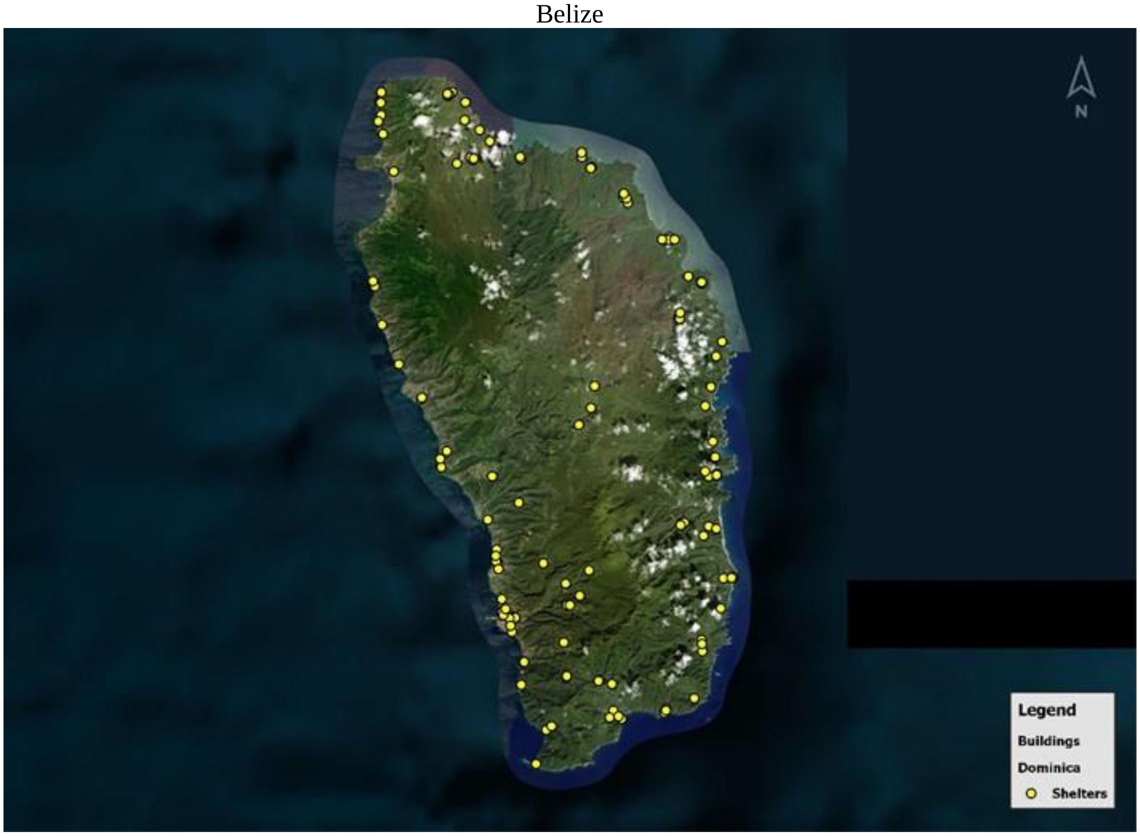
Sector	Subsector	Type	Components	Hazard
BUILDINGS	Schools, Hospitals, Government buildings, Emergency centers, Hospitality and hotels	Unreinforced masonry (URM)	Wall	Earthquake
			Roof	Wind
		Reinforced/concrete frame (RC/CF) with infill wall	Shape and figure	Earthquake
			Wall	Earthquake
			Roof	Wind
			Opening	Flood
		RC frame	Shape and figure	Earthquake
			Ductility	Earthquake
			External wall	Wind
		Steel frame	Stiffness	Earthquake, Wind
			Strength	Earthquake
			Roof	Wind
		Steel frame high-rise	Stiffness	Earthquake, Wind
			Ductility	Earthquake
			Cladding	Wind
		Wood frame	Roof	Wind
			Foundation	Wind, Flood
			Elevation	Flood
		All	Foundation	Liquefaction

Table 3-1. Analysis matrix for building infrastructure

3.2 Geographical distribution

The exposure information for critical buildings (i.e., schools, hospitals, government buildings, emergency buildings and hotels in this study) in selected Caribbean countries is presented in Figure 3-1. The collected information covers mostly site location for the selected countries, as the location of other critical buildings in these countries was not available. This type of geographical information is very useful for both disaster damage assessments performed on hazard maps and post-disaster response planning using regional transportation networks. For example, in Belize, the locations of four types of critical buildings are shown and the multiple access routes between these facilities (which is important for quick response and smooth recovery). These routes might be arranged beforehand by considering the spatial distribution of expected damage to buildings and roads due to a disaster. In addition, a complete package of exposure data of buildings is helpful to prioritize buildings in terms of disaster strengthening and financial planning. Preferably, information such as building occupancy, area, stories, materials, year built, occupants and so on, in addition to building location, should be included in the exposure data package.

Belize



Dominica

Grenada

Saint Lucia

Suriname

Figure 3-1. Critical building locations for selected Caribbean countries (Gosine, 2020a)

3.3 Typology

The buildings in focus in this study, schools, hospitals, government buildings, emergency buildings and hotels, are classified according to six structural types, as shown in Figure 3-2. The structural type usually refers to the building material, and brick/block masonry, concrete, steel and wood are used most typically for the buildings of those occupancies in the Caribbean. More specifically, unreinforced masonry (URM), reinforced/concrete frame (RC/CF) with infill wall, reinforced concrete (RC) frame, steel frame and wood frame are utilized for those buildings. Across these types, URM is more particularly vulnerable than others, as the disaster damage to this type of building is enormous in many countries. The structural types of buildings listed above are emphasized in this study.

URM

RC/CF with infill wall

RC frame

Steel frame

Steel frame high-rise

Wood frame

Figure 3-2. Building typology (various sources)

3.4 Vulnerable components

Per past disaster damage experiences, some vulnerable components have been observed in typical and recurrent failure modes, depending on structural type and hazard event. Those vulnerable components should be strengthened first to avoid severe damage and to make the buildings more resilient in future natural disasters. Those components and damage failures are focused on in this study. Typical failure modes due to earthquake shaking and liquefaction are shown in Figure 3-3 and representative damage due to hurricane winds and associated floods are presented in Figure 3-4.

URM Out-of-Plane and In-Plane wall damage due to shaking (Chin, 2012)

Soft/weak story collapse due to structural irregularities (Gibbs, 2012)

In-plane wall damage due to shaking (Clarke, 2018)

Column failure due to non-ductile detailing (Chin, 2012)

Joint failure due to non-ductile detailing (Rojas-Mercedes et al., 2020)

Ground failure due to liquefaction (Bakir, 2016)

Figure 3-3. Building damages due to earthquake shaking and liquefaction

The main element to resisting seismic force is typically a shear wall for URM or RC buildings. If the seismic capacity of a wall for in-plane or out-of-plane (OOP) is not sufficient, the building could be extensively damaged, and the failure mode could be very brittle (sudden). For avoiding brittle failure mode, modern design codes generally specify detailing rules (i.e., reinforcement restrictions, steel section limit, etc.) to make the elements more ductile. However, this kind of ductile detailing might not have been applied to the buildings, which were built without modern seismic design or code compliance in design or construction.

In that case, the building could suffer brittle damage without enough deformability. The structural shape of a building also affects the building's movement and response in an earthquake. If structural irregularities are observed, the building would suffer severe damage due to unbalanced load concentration or large deformation. For example, soft/weak stories, due to vertical discontinuity of wall or torsional irregularity from uneven horizontal location of wall, have caused critical damage to buildings in past earthquakes. Earthquake liquefaction causes large ground failure and the building typically loses foundational stability, as shown in the last picture of Figure 3-3.

Roof and wall connections are vulnerable components to wind hazard. Wind forces are imposed on buildings both horizontally and vertically. If the connection capacity is deficient for those forces, either or both of roof and wall elements could be blown away, and the building might suffer severe damage due to a lack of bearing elements, like a wall. Steel and wood buildings are relatively flexible structures because of the smaller sizes of columns and beams. This type of building is generally vulnerable to wind hazard; an appropriate stiffness should be added to resist strong wind forces. For mitigating flood damage (i.e., water inundation), elevating the building and/or protecting the building envelope from water inundation could represent possible improvements.

Roof and wall damage due to wind (Clarke, 2018)

Steel building damage due to wind (Gibbs, 2001)

Flooding in city (APESL, 2010)

Flooding in street (APESL, 2010)

Figure 3-4. Building damages due to hurricane winds and associated floods

The components listed in Table 3-2 caused severe damages to the building types discussed in this chapter in past events. These components are then assumed to be the major factors of damage and will be considered in this chapter.

Subsector	Type	Component	Vulnerability	Hazard
Schools, Hospitals, Government buildings,	URM	Wall	Out-of-plane capacity	Earthquake
			In-plane capacity	Earthquake
		Roof	Roof connection	Wind

Emergency centers, Hospitality and hotels	RC/CF with infill wall	Shape and figure	Structural irregularity	Earthquake
		Wall	Infill wall connection	Earthquake
		Roof	Roof connection	Wind
		Opening	Building envelope	Flood
	RC frame	Shape and figure	Structural irregularity	Earthquake
		Ductility	Non-ductile detailing	Earthquake
		External wall	External wall	Wind
	Steel frame	Stiffness	Stiffness	Earthquake, Wind
		Strength	Structural capacity	Earthquake
		Roof	Roof connection	Wind
	Steel frame high-rise	Stiffness	Stiffness	Earthquake, Wind
		Ductility	Non-ductile detailing	Earthquake
		Cladding	External curtain wall	Wind
	Wood frame	Roof	Roof connection	Wind
		Foundation	Foundation ties	Wind, Flood
		Elevation	Elevated construction	Flood
	All	Foundation	Lack of capacity	Liquefaction

Table 3-2. Vulnerable components of building infrastructure and corresponding hazards

3.5 Design codes

The Caribbean Uniform Building Code (CUBiC) was developed within and by the Caribbean community in 1985 and it includes structural design requirements for gravity load, wind load and seismic load (CARICOM, 1985). Since the degree of enforcement of this building code varies according to country, some countries of the Caribbean have adopted this code as part of the design process, but some treat this as a supplemental document, as shown in Figure 3-5 (Redmond, 2015). Some countries in the Caribbean have developed their own building code and use it as well as CUBiC, the International Building Code (IBC) and other modern design codes. In certain cases, foreign design consultants seem to apply their familiar design codes to structural design. On the other hand, a code-compliance process, like plan check, construction inspection and building permits, appears to not be guaranteed in many Caribbean countries, because of, for example, a lack of skilled inspectors, weak/inadequate enforcement mechanisms or less motivation for compliance. In this section, CUBiC and additional codes are investigated.

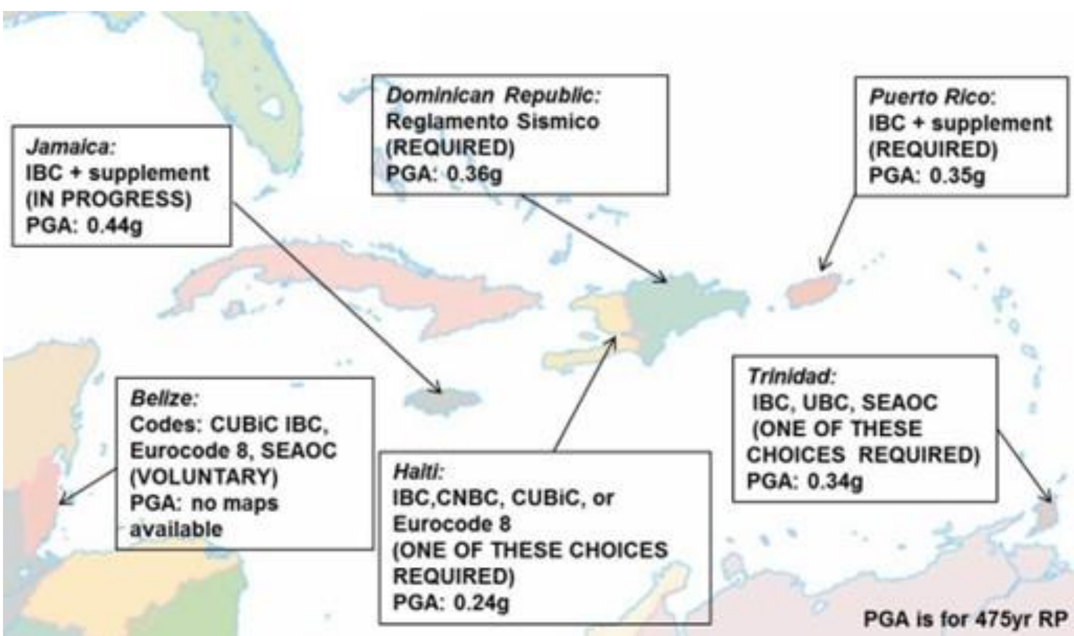


Figure 3-5. Building design codes of representative countries in the Caribbean (Redmond, 2015)

3.5.1 Seismic design

3.5.1.1 Seismic design in the Caribbean

In CUBiC Part 2, Section 3, the seismic design requirements are specified. Specific performance objectives for seismic design are not explicitly given, but the main objective provided is to protect human life and to reduce damage caused by earthquakes (Chin, 2008a). This seismic design approach observes a method controlled by displacements, inelastic deformations are accepted, and adequate ductility is required. This code has not been updated in accordance with engineering research accomplishments or earthquake experiences since it was published in 1985. For example, the current seismic hazard level should be updated from 1985 as it directly affects the seismic design force and, subsequently, building performance; however, this update has not occurred. Major design components for earthquakes specified in CUBiC are described as follows (Chin, 2008a; CUBiC, 1985).

3.5.1.1.1 Design seismic force

In the code, no seismic design response spectrum is specified nor any levels of seismic intensity (e.g., return period, effective PGA or seismicity) clarified. It appears that zonal coefficient, Z, is applied to each Caribbean country to calculate the seismic design force. The following formula is the equivalent seismic force for design in the equivalent static method of CUBiC:

Eq. 3-1 $V = ZCIKSW$

- V: The total lateral force of shear at the base
- Z: Numerical coefficient related to the seismicity of a region (see Table 3-3)
- C: Numerical coefficient, $C = 1/15\sqrt{T}$ (C need not exceed 0.12)
- T: Fundamental elastic period of vibration of the structure in seconds
- K: Numerical coefficient regarding ductility (see Table 3-4 and Table 3-5)
- I: Occupancy importance coefficient (see Table 3-6)
- S: Numerical coefficient for site-structure resonance
- $T/Ts \leq 1.0$; $S = 1.0 + T/Ts - 0.5(T/Ts)^2$
- $T/Ts > 1.0$; $S = 1.2 + 0.6(T/Ts) - 0.3(T/Ts)^2$
- Ts: Characteristic site period in seconds
- W: The total dead load and applicable portions of other loads

Country	Z value
Jamaica	0.75
Antigua	0.75
Saint Kitts and Nevis	0.75
Montserrat	0.75
Dominica	0.75
Saint Lucia	0.75
Saint Vincent and the Grenadines	0.50
Grenada	0.50
Barbados	0.375
NW Trinidad	0.75
Rest of Trinidad	0.50
Tobago	0.50
Guyana Essequibo	0.25
Rest of Guyana	0.00
Belize - areas within 100 km of southern border, i.e., including San Antonio and Punta Gorda, but excluding Middlesex, Pomona and Stann Crecil	0.75
Remainder of Belize	0.50

Table 3-3. Z values for various Caribbean countries (CUBiC, 1985)

Item	Description	*K* factor
1	Ductile frames with an adequate number of possible plastic beam hinges	0.8
2	Ductile frames with an inadequate number of possible plastic beam hinges	1.0
3	Ductile coupled shear walls	0.8
4	Two or more parallel and approximately symmetrically arranged cantilever shear walls	1.0
5	Single ductile cantilever shear walls	1.2
6	Shear walls not designed for ductile flexural yielding but having the ability to dissipate a significant amount of seismic energy	1.6
7	Buildings with diagonal bracing capable of plastic deformation in tension only a) Single story b) Two or three stories c) More than three stories	 2.0 2.5 or by special study by special study
8	a) Buildings in which part of the horizontal load is resisted by item 7 bracing and in part by an item 1 or item 2 frame b) Buildings with diagonal bracing capable of plastic deformation in both tension and compression	1.6 or by special study 1.6 or by special study
9	Small tanks on the ground	2.0

Table 3-4. *K* factors for various structural types, steel and concrete (CUBiC, 1985)

Item	Description	*K* factor
B1	Shear walls or diaphragm (a) Ductile	 1.0
	(b) Ductile and stiffened with elastomeric adhesive (c) Limited ductility fixed with elastomeric adhesive	1.0 1.2
B2	Moment resisting frames (a) Ductile with an adequate number of possible plastic beam hinges (b) Ductile with an inadequate number of possible plastic beam hinges (c) As for item B2(a) but with connections of limited ductility (d) As for item B2(b) but with connections of limited ductility (e) Non-ductile	 1.2 1.5 1.5 1.7 2.4
B3	Diagonally braced with timber members capable of acting as struts or ties: (a) With ductile end connections (b) With end connections having limited ductility	 1.7 2.0

Table 3-5. *K* factors for various structural types, timber (CUBiC, 1985)

Building class	Description	*I*
I	These are essential facilities required for use in the aftermath of a major earthquake, e.g. hospitals, fire stations, communication centers etc.	1.5
II	These are public buildings and buildings which accommodate large numbers of' people, e.g. cinemas, theatres, schools, defense establishments etc.	1.25
III	All other buildings not included in Class I or Class II above.	1.0

Table 3-6. *I* factors for various structural types, steel and concrete (CUBiC, 1985)

3.5.1.1.2 Structural type

Nine structural types for concrete and steel buildings and three structural types for timber buildings are specified, and design seismic force is adjusted (reduced or increased) by factor *K* according to this type due to ductile performance.

Frame Type: Ductile steel, concrete, timber.

Dual Type: Frame + Wall combination. Frame with 25% of shear demand; steel, concrete, masonry, timber.

Wall Type: Either concrete, masonry or plywood walls or steel or timber braced frames.

Cantilever Type: Inverted Pendulum.

Others Type: None of the above. (CUBiC, 1985)

3.5.1.1.3 Structural irregularity

The code requires a dynamic analysis method if the building has large irregularities in plan and/or elevation. In particular, buildings classified in Importance Groups I and II shall be analyzed by dynamic methods when:

(a) The seismic force resisting system does not have the same configuration in all stories and in all floors.

(b) The floor masses differ by more than 30% in adjacent floors.

(c) The cross-sectional areas and moments of inertia of structural members differ by more than 30% in adjacent stories. (CUBiC, 1985)

3.5.1.1.4 Drift limitation

The lateral drift due to a design seismic force is limited as follow:

Lateral deflections or drift of a story relative to its adjacent stories shall not exceed 0.005 times the story height unless it can be demonstrated that greater drift can be tolerated. The displacement calculated from the application of the required lateral forces shall be multiplied by 1.0/K to obtain the drift. The ratio 1.0/K shall not be less than 1.0. (CUBiC, 1985)

3.5.1.2 Seismic design in Trinidad and Tobago

TTS 599: Guide to the Design and Construction of Small Buildings was developed in Trinidad and Tobago in 1997 because more than 80% of buildings in Trinidad and Tobago are small buildings, like low-rise building (Thomson, 2018). This document was established by referring to Part 5 of CUBiC "Small Buildings and Pre-fabricated Construction," and International Design Codes, and therefore the basic concepts of seismic design, such as performance objectives or design approach, are basically the same as that of CUBiC. However, the document has not been formally adopted by approval agencies, even though its latest edition was published in 2006. This lack of official acceptance means a formal and accepted national building code does not exist in Trinidad and Tobago (Thomson, 2018). Based on technical communications with local engineers and researchers, it was determined that the buildings in Trinidad and Tobago are generally designed for seismic forces based on empirical engineering approach or major international building codes. The main components of this design guideline are as follows (Chin, 2008b).

3.5.1.2.1 Design seismic force

A simplified formula for seismic design force is presented in this guideline. The following formula is based on the equivalent seismic force for design in the equivalent static method of CUBiC.

$$\text{Eq. 3-2} \quad V = 0.05 \times S \times W$$

- V: The total lateral force op shear at the base
- 0.05: Integrated value of Z = ground acceleration (Z value), C = amplification factor due to structure frequency, I = Importance factor =1 in this guideline and K = Ductility factor
- S: Site factor S = 1 for good soil (rock, gravel), S = 1.2 for softer material (clay, fill), S = 1.5 for deep alluvial deposits, S = 2.5 maximum for reclaimed land and saturated soils (due to the amplification factor)
- W: The total dead load and applicable portions of other loads

3.5.1.2.2 Earthquake resistant construction

Not only shaking, but also liquefaction is considered as an earthquake influence in this guideline.

> *General: Trinidad and Tobago is in an earthquake zone and has experienced varying degrees of damage due to earthquakes. It is therefore essential that buildings are designed and constructed so that they have some resistance to the shaking or lateral forces produced by earthquakes.*

> *Effect of soil: The type of soil at the site may have a significant effect upon the resistance of the building to an earthquake. However for buildings within the scope of this code the effect of the soil type is not so significant provided that the building is not constructed on loose saturated sands, which may liquefy during an earthquake and cause collapse of the building.*

3.5.1.2.3 Lateral load design

The basic approach regarding seismic force transfer is described, and a load path concept from diaphragm-to-foundation through collector and vertical element (e.g., wall, brace, etc.) is clarified.

> *Diaphragm: Floor, roof or ceiling assemblies may be constructed with the necessary stiffness and load path continuity to distribute lateral loads (wind and earthquake) to lateral support subsystems. In this role, floor, roof or ceiling surface act as horizontal beams (also called a diaphragm) spanning lateral supports points. Use of floor, roof or ceiling assembly, as a diaphragm requires both strength and stiffness properties and development of connections to transfer the diaphragm force.*

> *Shear panel: Concrete wall (A shear panel is a portion or section of a 150mm exterior wall that performs the function of resisting lateral earthquake or wind forces and Timber wall bracing.*

3.5.1.2.4 Load factors

Load combinations for gravity and earthquake design are specified in Table 3-7.

Combination	Dead	Live	Earthquake
Gravity	1.4	1.7	0
Earthquake (a)	1.05	1.275	+/- 1.4025
Earthquake (b)	0.9	0	+/- 1.43

Table 3-7. *I* Load combinations for design (Chin, 2008b)

3.5.2 Wind design

3.5.2.1 Wind design in the Caribbean

Sections of CUBiC Part 2 prescribe the structural design requirements of buildings for wind force. As with the seismic design component of CUBiC, explicit performance objectives are not expressed, but the main objectives provided are to protect the life of occupants and to reduce any damage due to wind load (Suite, 2008). Since the wind design component of CUBiC has not been updated since 1985, the design wind pressure is quite different from current modern codes like IBC or ASCE 7 (Gibbs, 2012). If the basic wind speed to calculate the design wind pressure is changed, the demand for building wind design will be directly affected. Primary components of wind design in CUBiC are shown as follows (Gibbs, 1998) (Suite, 2008).

- Concept and limitations

> *This document describes the action of wind on structures and methods for calculating characteristic values of wind loads for use in designing buildings, towers, chimneys, bridges and other structures as well as their components and appendages. These loads will be suitable for use in conjunction with other ISO Load Standards and with ISO 2394 – General Principles on Reliability for Structures.*

3.5.2.1.1 Design wind pressure (simplified procedure)

The formula below is prepared for calculating design wind pressure based on wind velocity and several factors. The height of wind is based on 15 m above ground, the average period is 10 minutes mean velocity pressure and the return period of wind velocity is 50 years. The following formula is for the simplified procedure.

$$\text{Eq. 3-3 } W = (q_{ref})(C_{exp})(C_{shp})(C_{dyn})$$

- W: Wind force per unit area normal to the surface of the structure
- q_{ref}: Reference velocity pressure (see Table 3-8 & Table 3-9 and Figure 3-6)
- $q = (1/2)\rho V^2$, ρ is air density, V is wind velocity
- C_{exp}: Exposure factor
- C_{shp}: Aerodynamic shape factor
- C_{dyn}: Dynamic response factor

Location	**Wind pressure (kPa)**			**Wind speed (m/sec)**
	q_{ref} (q_{50})	**q_{10}**	**q_{100}**	**V_{ref}**
Guyana	0.20	0.05	0.35	18.0
Trinidad-South	0.25	0.05	0.40	20.0
Trinidad-North	0.40	0.10	0.60	25.5
Tobago	0.47	0.15	0.65	28.0
Grenada	0.60	0.25	0.80	31.5
Barbados	0.70	0.30	0.90	34.2
Saint Vincent and the Grenadines	0.73	0.35	0.93	35.0
Saint Lucia	0.76	0.36	0.95	35.5
Dominica	0.85	0.42	1.06	37.5
Montserrat	0.83	0.40	1.07	37.2
Antigua	0.82	0.39	1.05	37.0
Saint Kitts and Nevis	0.83	0.38	1.07	37.2
Jamaica	0.80	0.40	1.00	36.5
Belize-North	0.78	0.38	1.00	36.0
Belize-South	0.55	0.26	0.70	30.5

Table 3-8. Reference wind velocity pressures for the Caribbean (CUBiC, 1985)

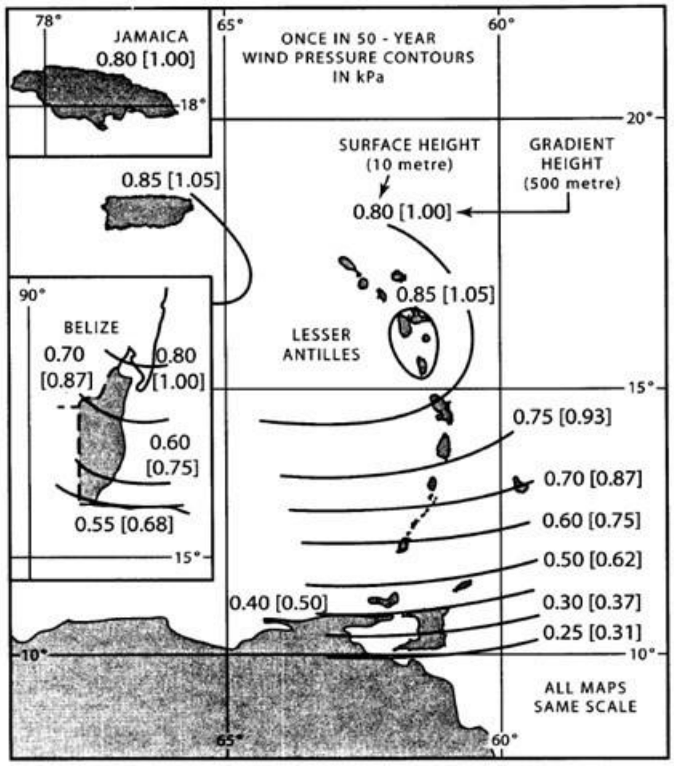


Figure 3-6. Regional map of design wind pressure contours based on CUBiC (Gibbs, 1998)

q_{ref} (kPa)	V peak (m/sec) – 10 meters			
	Average time			
10 min.	10 min.	1 hr.	1 min. (or "fastest mile")	3 sec.
0.30	22.4	21	27	33
0.40	25.8	25	31	39
0.50	28.9	27	35	43
0.60	31.6	30	38	47
0.70	34.2	32	41	51
0.80	36.5	35	44	55
0.90	38.7	37	47	58
1.00	40.8	39	50	61
1.10	42.8	41	52	64
1.20	44.7	43	54	67
1.30	46.5	44	56	70
1.40	48.3	46	58	73
1.50	50.0	48	61	75

Table 3-9. Relationship between reference velocity pressure q_{ref} and peak wind speeds over short time intervals in open terrain (CUBiC, 1985)

3.5.2.1.2 Wind design actions

Four major actions due to wind force are specified for consideration in the wind design of structures.

(a) *Excessive force or instability.*

(b) *Excessive deflection.*

(c) *Repeated dynamic forces causing fatigue.*

(d) *Aeroelastic instability.*

3.5.2.1.3 Method of analysis

Three methods of analysis are prepared in CUBiC: simplified procedure, detailed procedure and experimental procedure.

> *Simplified procedure: This method is intended for the design of normal structures and can be used for the wind design of the main structural system if (a) the structure is less than 15 m in height above ground, (b) the structure is not usually exposed for any wind direction, that is, it is not situated near a hill crest or head land, (c) the structure is relatively rigid. Deflection under wind loading less than 1/500.H where H = height of structure.*
>
> *Detailed procedure: This method is principally of assistance in assessing the dynamic response of the structure, the influence of unusual exposure and the characteristics of more complex aerodynamic shapes. Structures sensitive to wind include those that are particularly flexible, slender, lightweight or tall.*
>
> *Experimental procedure: This is recommended when assessing the influence of unusual exposure and the characteristics of more complex aerodynamic shapes as well as structures sensitive to wind including those that are particularly flexible, slender, lightweight or tall and structures of unusual geometry, which give rise to unexpectedly large responses to wind. It may be necessary to conduct supplementary studies by experts in the field. These tests may include wind tunnel tests.*

3.5.2.2 Wind design in Dominica

Based on a (draft) physical planning building code document, the wind design approach and design wind pressure for buildings in Dominica refer to CUBiC (Dominica, 1996) and therefore, it is anticipated that the current components for wind design for the Dominica building code also conform to CUBiC. A guideline for housing standards for hurricane hazards in Dominica was developed as a response to the enormous damage due to Hurricane Maria in 2017 (Dominica, 2018). This guideline, based on a specification-based approach, includes typical wind pressures on housing (see Figure 3-7), general failure modes due to wind and flood, and recommended design procedures for building shape, foundations, walls, connections, roofs, retrofit and maintenance. However, any specific wind load for housing design are not included.

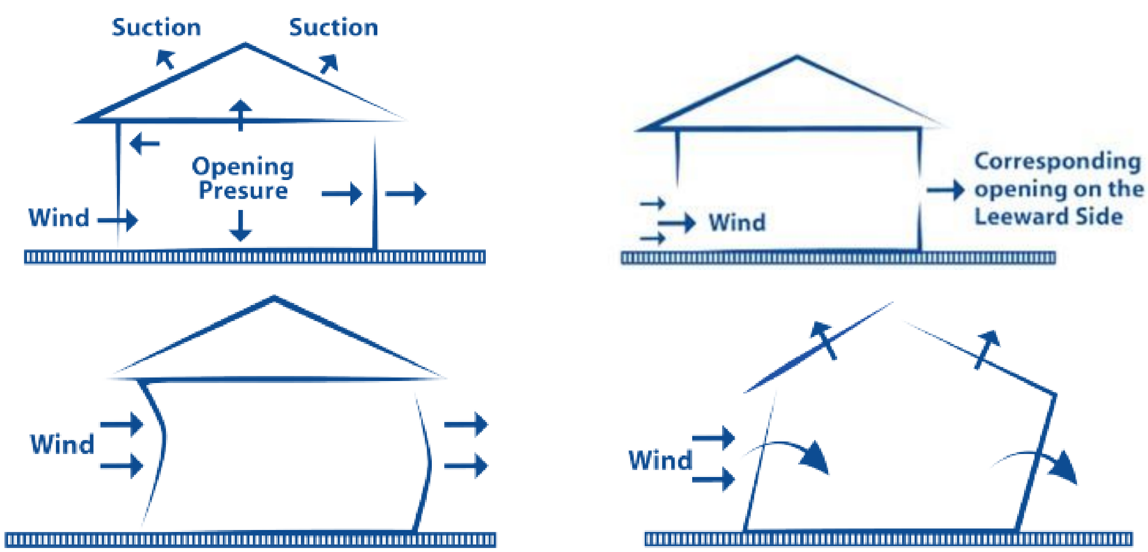


Figure 3-7. Typical wind pressure on housing (Dominica, 2018)

3.5.3 Flood design

3.5.3.1 Flood design by ASCE 7-16

It is assumed that the subject countries use flood design methodologies commonly used throughout the world (but taking into consideration each country's circumstances), as no flood design codes for buildings were found. In this section, therefore, the design approach for floods prescribed in ASCE 7-16 (ASCE, 2017a) is briefly introduced, as the International Building Code (ICC 2017 – widely used in the Caribbean) and ASCE 24-14 (ASCE 2015 – document focusing on flood design) refer to ASCE 7-16.

The base flood specified in ASCE 7-16 for building design is the flood having a 1% chance of being equaled or exceeded in any given year. So, the flood used for design needs to be compared to this base flood with the site-specific flood hazard (ASCE, 2017a). The major terms for flood design are listed below.

> *Base flood: The flood having a 1% chance of being equaled or exceeded in any given year.*
>
> *Design flood: The greater of the following two flood events: (1) the base flood, affecting those areas identified as special flood hazard areas on the community's FIRM; or (2) the flood corresponding to the area designated as a flood hazard area on a community's flood hazard map or otherwise legally designated.*
>
> *Base flood elevation (BFE): The elevation of flooding, including wave height, having a 1% chance of being equaled or exceeded in any given year.*
>
> *Design flood elevation (DFE): The elevation of the design flood, including wave height, relative to the datum specified on a community's flood hazard map.*
>
> *Flood hazard area: The area subject to flooding during the design flood.*
>
> *Flood hazard map: The map delineating flood hazard areas adopted by the Authority Having Jurisdiction.*
>
> *Flood insurance rate map (FIRM): An official map of a community on which the Federal Insurance and Mitigation Administration has delineated both special flood hazard areas and the risk premium zones applicable to the community.*
>
> *Special flood hazard area (area of special flood hazard): The land in the floodplain subject to a 1% or greater chance of flooding in any given year. These areas are delineated on a community's FIRM as A-Zones (A, AE, A1-30, A99, AR, AO, or AH) or V-Zones (V, VE, VO, or V1-30).* (ASCE, 2017a)

In this code, it is specified that structural elements of buildings or structures shall be designed, constructed, connected, and anchored to resist flotation, collapse, and permanent lateral displacement due to the action of flood loads associated with the design flood and other loads in accordance with load combinations (ASCE, 2017a). Several loads during flooding are prescribed for design use, such as hydrostatic load, hydrodynamic load, wave load and impact loads. As an example, the dynamic static and total pressure distributions against a wall are shown in Figure 3-8.

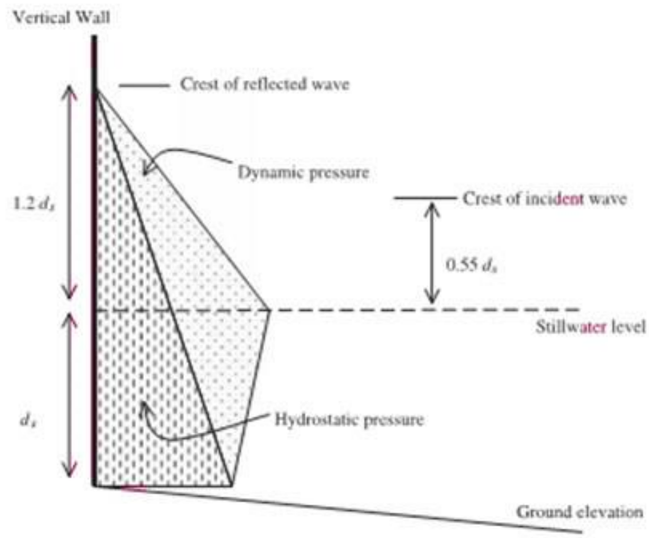


Figure 3-8. Dynamic static and total pressure distributions against wall (ASCE, 2017a)

3.5.3.2 Flood design in Jamaica

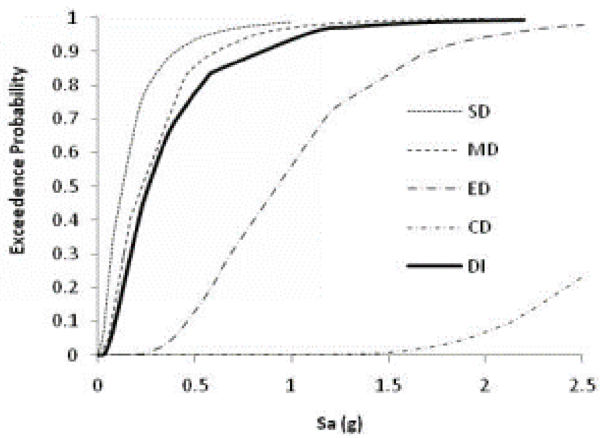
From a document presenting the Jamaica National Building Act (Da Costa, 2018), the flood design for buildings in Jamaica refers to the International Building Code (ICC, 2017) and the latest ASCE 7 (ASCE, 2017a). The contents of the flood design methodology of ASCE 7-16 can be seen in Section 3.5.3.1.

3.6 Fragility functions

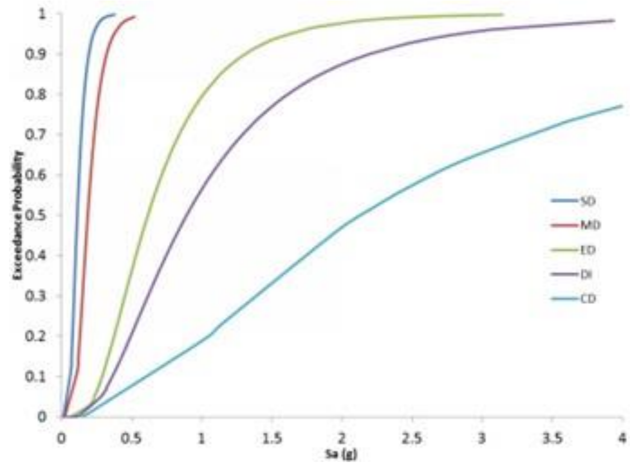
Fragility functions are broadly applied to assess the disaster capacity of structures and the disaster risk due to structural damage. Fragility functions generally express the probability of being in or exceeding a given damage state (e.g., slight, moderate, extensive, complete) corresponding to disaster intensity (e.g., acceleration, velocity, displacement, etc.). The function is modeled as a cumulative lognormal distribution and is characterized by a median value and lognormal standard deviation of disaster intensity. Since this function is essentially based on structural damage states, it is suitable to compare the structural damage of the existing structure and the retrofitted structure discussed in this study.

For the Caribbean, there are a few research studies on the fragility functions of local buildings. Clarke and Carey performed an analytical procedure to evaluate the seismic fragility functions of low-rise URM residential buildings in Trinidad and Tobago, as shown in Figure 3-9 (Clarke, 2017) (Carey & Clarke, 2012). Hancilar took an empirical approach based on the actual damage records from the 2010 Haiti earthquake in order to estimate the seismic fragility functions of typical buildings in Haiti; see Figure 3-9 (Hancilar et al., 2013). However, both studies are for existing buildings and any comprehensive research on fragility functions of both pre- & post-retrofitted buildings and other disasters in the Caribbean are not available.

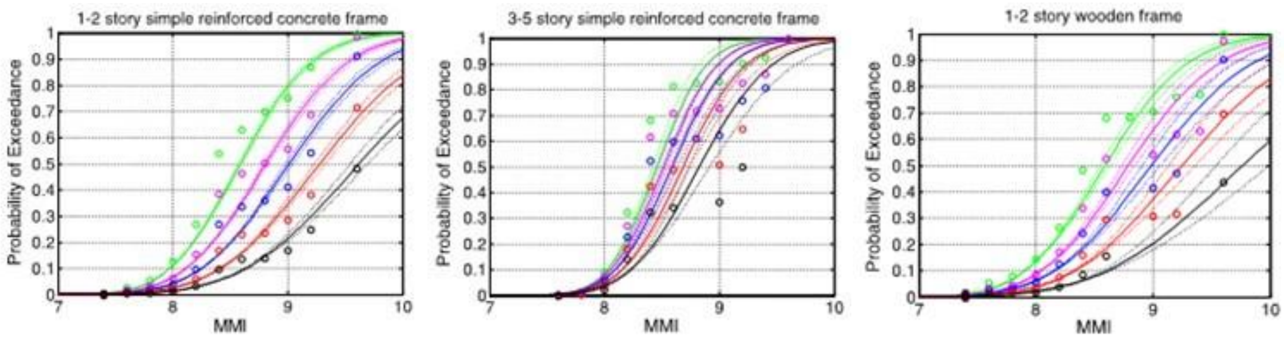
The Federal Emergency Management Agency (FEMA) in the U.S. developed comprehensive documents on disaster risk assessment, including fragility functions, and provides various types of fragility functions for buildings, including essential facilities like emergency centers, hospitals and schools (FEMA 2013a)(FEMA, 2013b)(FEMA, 2013c). In these documents, the fragility functions are classified in several groups according to structural type and design code level (i.e., design quality based on primarily the design era). Under these circumstances, the more comprehensively-developed fragility functions from FEMA are utilized in this section.



Trinidad and Tobago (Clarke, 2017)



Trinidad and Tobago (Carey & Clarke, 2012)



Haiti (Hancilar et al., 2013)

Figure 3-9. Seismic fragility function studies in the Caribbean

3.6.1 Earthquake ground shaking: FEMA Hazus fragility functions

For fragility functions for earthquake shaking, FEMA Hazus (FEMA, 2013a) provides a comprehensive package of this function for buildings according to structural types and height classes. In addition, FEMA Hazus developed several ranks of functions according to design level, which relates to seismic design code and era built. For example, the fragility function of a building designed with high-level codes is evaluated more robustly than a building designed with low-level codes, even if the buildings are the same structural type. This consideration is very useful to estimate an improvement degree resulting from a retrofit measure because an improvement is usually conducted to meet the performance objectives of the high-level code.

In this section, for simplicity, it is assumed that the existing buildings (before retrofitting) were designed based on low-level codes and that the retrofitted buildings are strengthened to possess the same seismic performance specified by a high-level code. The performance improvement for buildings in an earthquake is then discussed based on two fragility functions, according to existing or retrofitted condition. The fragility functions presented in Hazus are used for this performance comparison, but Hazus also provides a conversion procedure for fragility functions per specific location or soil type. Therefore, the functions can be adjusted by procedure if any specific locations or countries are focused on in a future study. In the following subsections, the seismic fragility functions are described for the representative structural types of buildings.

3.6.1.1 URM buildings

3.6.1.1.1 Overview

For unreinforced masonry (URM) buildings, the description in Hazus for the structural type, four damage states and fragility functions of existing and retrofitted buildings are presented as follows, and an expected degree of retrofit improvement is discussed.

3.6.1.1.2 Structural type for URM

> *Unreinforced masonry (URM): These buildings include structural elements that vary depending on the building's age and, to a lesser extent, its geographic location. In buildings built before 1900, the majority of floor and roof construction consists of wood sheathing supported by wood framing. In large multistory buildings, the floors are cast-in-place concrete supported by the unreinforced masonry walls and/or steel or concrete interior framing. In unreinforced masonry constructed after 1950 (outside California) wood floors usually have plywood rather than board sheathing. In regions of lower seismicity, buildings of this type constructed more recently can include floor and roof framing that consists of metal deck and concrete fill supported by steel framing elements. The perimeter walls, and possibly some interior walls, are unreinforced masonry. The walls may or may not be anchored to the diaphragms. Ties between the walls and diaphragms are more common for the bearing walls than for walls that are parallel to the floor framing. Roof ties usually are less common and more erratically spaced than those at the floor levels. Interior partitions that interconnect the floors and roof can reduce diaphragm displacements.*

3.6.1.1.3 Structural damage states description for URM

FEMA Hazus (FEMA, 2003a) defines a number of damage states. The damage states for URM buildings are summarized in Table 3-10.

Damage State	State	Description
DS1	None	No observable damage
DS2	Slight	Diagonal, stair-step hairline cracks on masonry wall surfaces; larger cracks around door and window openings in walls with large proportion of openings; movements of lintels; cracks at the base of parapets
DS3	Moderate	Most wall surfaces exhibit diagonal cracks; some of the walls exhibit larger diagonal cracks; masonry walls may have visible separation from diaphragms; significant cracking of parapets; some masonry may fall from walls or parapets.
DS4	Extensive	In buildings with relatively large area of wall openings most walls have suffered extensive cracking. Some parapets and gable end walls have fallen. Beams or trusses may have moved relative to their supports.
DS5	Complete	Structure has collapsed or is in imminent danger of collapse due to in-plane or out-of-plane failure of the walls. Approximately 15% of the total area of URM buildings with complete damage is expected to be collapsed.

Table 3-10. URM building damage states (adapted from FEMA, 2003a)

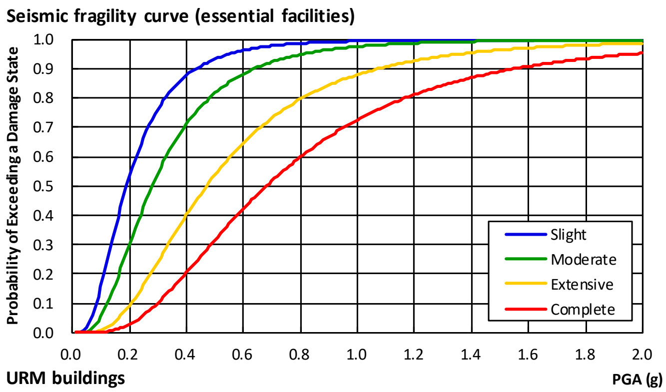
3.6.1.1.4 Fragility functions for URM

In Table 3-11, the parameters of lognormal distribution for the fragility curves of existing and retrofitted buildings are listed. For the retrofitted URM building, it is assumed that it would be the same performance level of a reinforced masonry building, because URM buildings are out of scope in high-level building codes and therefore should be considered reinforced masonry. Based on these parameters, the fragility curves and related probability mass function for existing and retrofitted URM buildings are presented in Figure 3-10 and Figure 3-11.

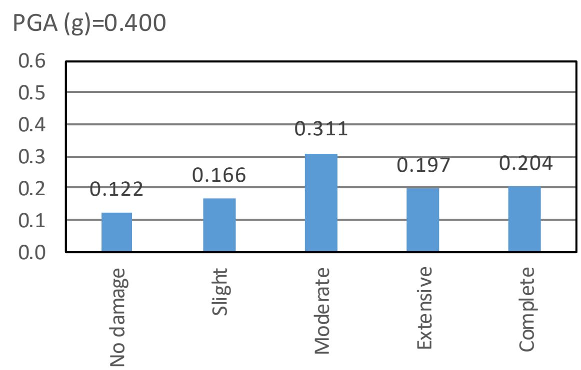
As shown in the fragility functions, the curves of all damage states are largely improved by seismic retrofit implementation. For an earthquake with a PGA of 0.4 g (an average value expected from an earthquake occurring in the Caribbean once in 475 years), the probability of "No damage" is 12% for the existing building and is 48% for the retrofitted building; the probability of exceeding "Extensive damage" is 40% for the existing building and is 2% for the retrofitted building. Thus, seismic retrofit can be expected to greatly reduce the probability of seismic damage to URM buildings.

Type	**Status**	**Median (g); PGA**				**Lognormal standard deviation (g)**
		Slight	**Moderate**	**Extensive**	**Complete**	
URM	Existing	0.19	0.28	0.47	0.68	0.64
	Retrofitted	0.39	0.65	1.52	2.53	0.64

Table 3-11. Parameters of seismic fragility functions for URM buildings (FEMA, 2013a)

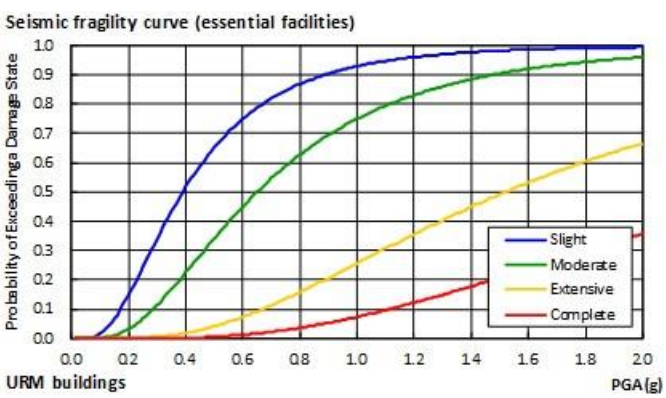


Seismic fragility functions

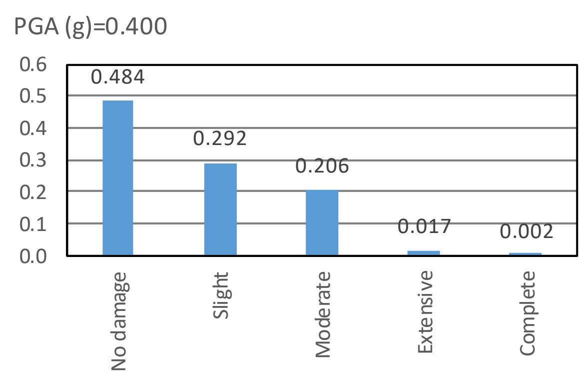


Probability mass function at PGA=0.4g

Figure 3-10. Seismic fragility functions & probability mass function at PGA=0.4g, Existing URM buildings



Seismic fragility functions



Probability mass function at PGA=0.4g

Figure 3-11. Seismic fragility functions & probability mass function at PGA=0.4g, Retrofitted URM buildings

3.6.1.2 RC/CF with infill wall buildings

For reinforced/concrete frame (RC/CF) with infill wall buildings, the description in Hazus for the structural type, four damage states and fragility functions of existing and retrofitted buildings are presented as follows, and an expected degree of retrofit improvement is discussed.

3.6.1.2.1 Structural type for RC/CF with infill wall

> *Concrete frame with infill wall (C3): This is one of the older types of buildings. The infill walls usually are offset from the exterior frame members, wrap around them, and present*

a smooth masonry exterior with no indication of the frame. Solidly infilled masonry panels, when they fully engage the surrounding frame members (i.e. lie in the same plane), may provide stiffness and lateral load resistance to the structure. In these buildings, the shear strength of the columns, after cracking of the infill, may limit the semi-ductile behavior of the system.

3.6.1.2.2 Structural damage states description for RC/CF with infill wall

FEMA Hazus (FEMA, 2003a) defines a number of damage states. The damage states for RC/CF with infill wall buildings are summarized in Table 3-12.

Damage State	State	Description
DS1	None	No observable damage
DS2	Slight	Diagonal (sometimes horizontal) hairline cracks on most infill walls; cracks at frame-infill interfaces.
DS3	Moderate	Most infill wall surfaces exhibit larger diagonal or horizontal cracks; some walls exhibit crushing of brick around beam-column connections. Diagonal shear cracks may be observed in concrete beams or columns
DS4	Extensive	Most infill walls exhibit large cracks; some bricks may dislodge and fall; some infill walls may bulge out-of-plane; few walls may fall partially or fully; few concrete columns or beams may fail in shear resulting in partial collapse. Structure may exhibit permanent lateral deformation.
DS5	Complete	Structure has collapsed or is in imminent danger of collapse due to a combination of total failure of the infill walls and non-ductile failure of the concrete beams and columns. Approximately 15% (low-rise), 13% (mid-rise) or 5% (high-rise) of the total area of C3 buildings with complete damage is expected to be collapsed.

Table 3-12. RC/CF with infill building damage states (adapted from FEMA, 2003a)

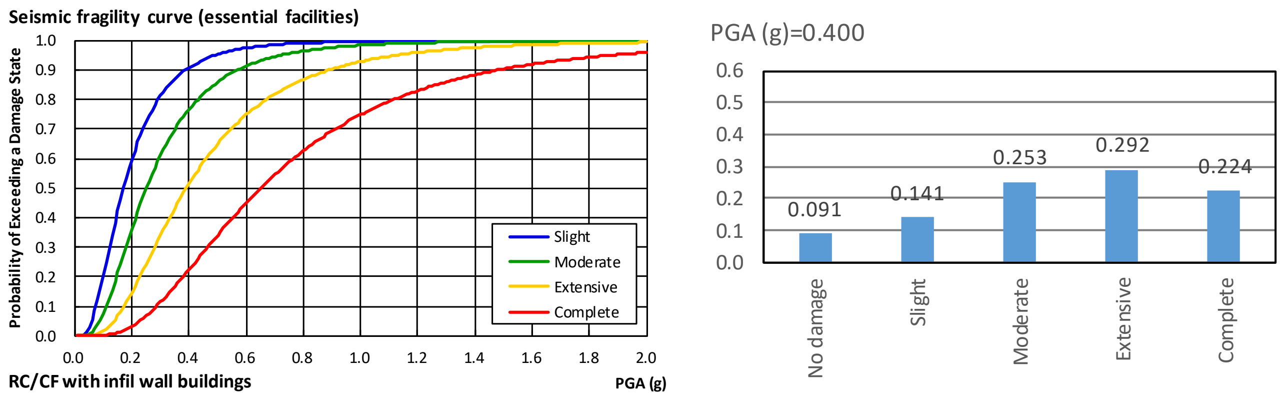
3.6.1.2.3 Fragility functions for RC/CF with infill wall

In Table 3-13, the parameters of lognormal distribution for the fragility curves of existing and retrofitted buildings are listed. For the retrofitted RC/CF with infill wall building, it is assumed that it would be the same performance level of a reinforced concrete frame building, as RC/CF with infill wall buildings are out of scope in high-level building codes and therefore should be considered reinforced concrete frame. Based on these parameters, the fragility curves and relating probability mass function for existing and retrofitted RC/CF with infill wall buildings are presented in Figure 3-12 and Figure 3-13.

As shown in the fragility functions, the curves of all damage states are largely improved by seismic retrofit implementation. For an earthquake with a PGA of 0.4 g (an average value expected from an earthquake occurring in the Caribbean once in 475 years), the probability of “No damage” is 9% for the existing building and 31% for the retrofitted building; the probability of exceeding “Extensive damage” is 52% for the existing building and 6% for the retrofitted building. Thus, seismic retrofit can be expected to greatly reduce the probability of seismic damage to RC/CF with infill wall buildings.

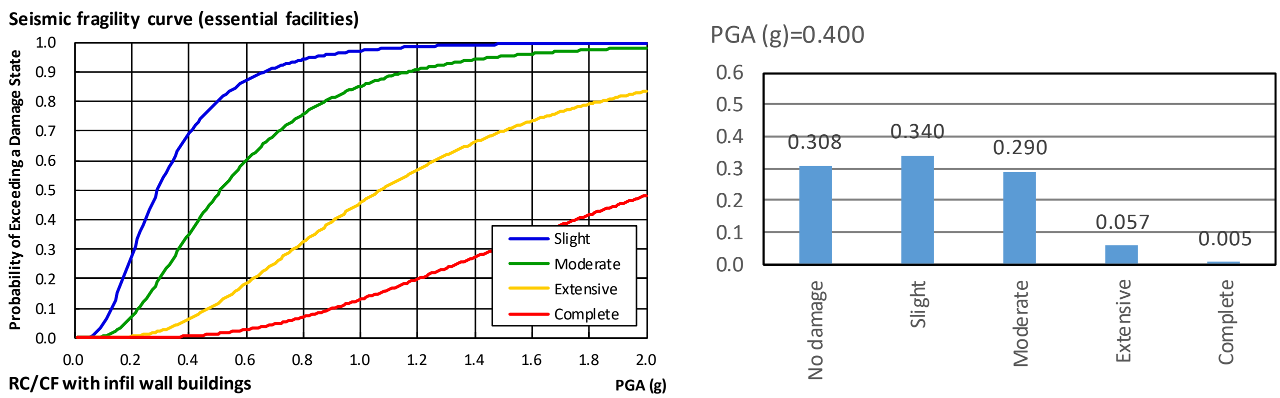
Type	Status	Median (g); PGA				Lognormal standard deviation (g)
		Slight	Moderate	Extensive	Complete	
RC/CF with infill wall	Existing	0.17	0.25	0.39	0.65	0.64
	Retrofitted	0.29	0.51	1.07	2.06	0.64

Table 3-13. Parameters of seismic fragility functions for RC/CF with infill wall buildings (FEMA, 2013a)



Seismic fragility functions Probability mass function at PGA=0.4g

Figure 3-12. Seismic fragility functions & probability mass function at PGA=0.4g, Existing RC/CF with infill wall buildings



Seismic fragility functions Probability mass function at PGA=0.4g

Figure 3-13. Seismic fragility functions & probability mass function at PGA=0.4g, Retrofitted RC/CF with infill wall buildings

3.6.1.3 RC frame buildings

For reinforced concrete (RC) frame buildings, the description in Hazus for the structural type, four damage states and fragility functions of existing and retrofitted buildings are presented as follows, and an expected degree of retrofit improvement is discussed.

3.6.1.3.1 Structural type for RC frame

> *Reinforced concrete frame (C1): These buildings are similar to steel moment frame buildings except that the frames are reinforced concrete. There are a large variety of frame systems. Some older concrete frames may be proportioned and detailed such that brittle failure of the frame members can occur in earthquakes leading to partial or full collapse of the buildings. Modern frames in zones of high seismicity are proportioned and detailed for ductile behavior and are likely to undergo large deformations during an earthquake without brittle failure of frame members and collapse.*

3.6.1.3.2 Structural damage states description for RC frame

FEMA Hazus (FEMA, 2003a) defines a number of damage states. The damage states for RC frame buildings are summarized in Table 3-14.

Damage State	State	Description
DS1	None	No observable damage

Damage State	State	Description
DS2	Slight	Flexural or shear type hairline cracks in some beams and columns near joints or within joints.
DS3	Moderate	Most beams and columns exhibit hairline cracks. In ductile frames some of the frame elements have reached yield capacity indicated by larger flexural cracks and some concrete spalling. Non-ductile frames may exhibit larger shear cracks and spalling.
DS4	Extensive	Some of the frame elements have reached their ultimate capacity indicated in ductile frames by large flexural cracks, spalled concrete and buckled main reinforcement; non-ductile frame elements may have suffered shear failures or bond failures at reinforcement splices, or broken ties or buckled main reinforcement in columns which may result in partial collapse.
DS5	Complete	Structure is collapsed or in imminent danger of collapse due to brittle failure of non-ductile frame elements or loss of frame stability. Approximately 13% (low-rise), 10% (mid-rise) or 5% (high-rise) of the total area of C1 buildings with complete damage is expected to be collapsed.

Table 3-14. RC frame building damage states (adapted from FEMA, 2003a)

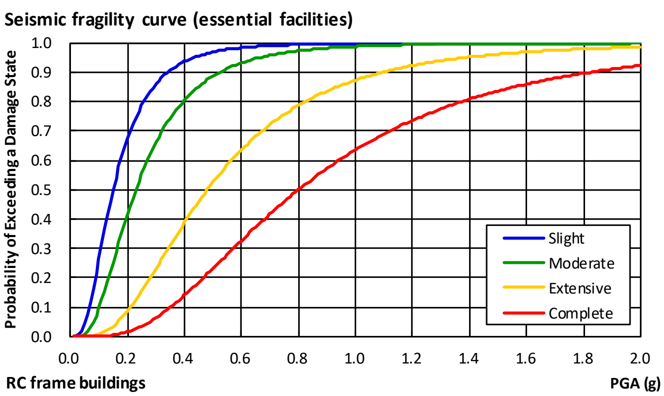
3.6.1.3.3 Fragility functions for RC frame

In Table 3-15, the parameters of lognormal distribution for the fragility curves of existing and retrofitted RC frame buildings are listed. Based on these parameters, the fragility curves and relating probability mass function for existing and retrofitted RC frame buildings are presented in Figure 3-14 and Figure 3-15.

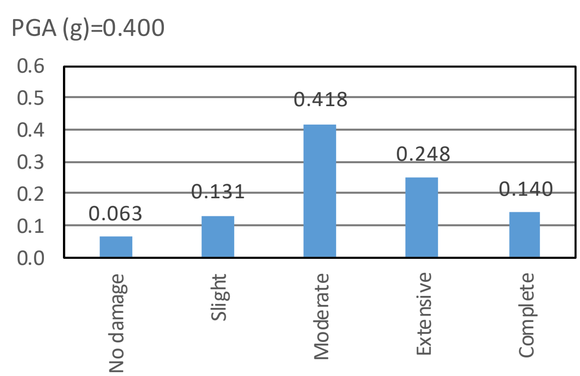
As shown in the fragility functions, the curves of all damage states are largely improved by seismic retrofit implementation. For an earthquake with a PGA of 0.4 g (an average value expected from an earthquake occurring in the Caribbean once in 475 years), the probability of "No damage" is 6% for the existing building and 12% for the retrofitted building; the probability of exceeding "Extensive damage" is 39% for the existing building and 7% for the retrofitted building. Thus, seismic retrofit can be expected to greatly reduce the probability of seismic damage to RC frame buildings.

Type	Status	Median (g); PGA				Lognormal standard deviation (g)
		Slight	Moderate	Extensive	Complete	
RC frame	Existing	0.15	0.23	0.48	0.80	0.64
	Retrofitted	0.19	0.36	1.02	2.48	0.64

Table 3-15. Parameters of seismic fragility functions for RC frame buildings (FEMA, 2013a)

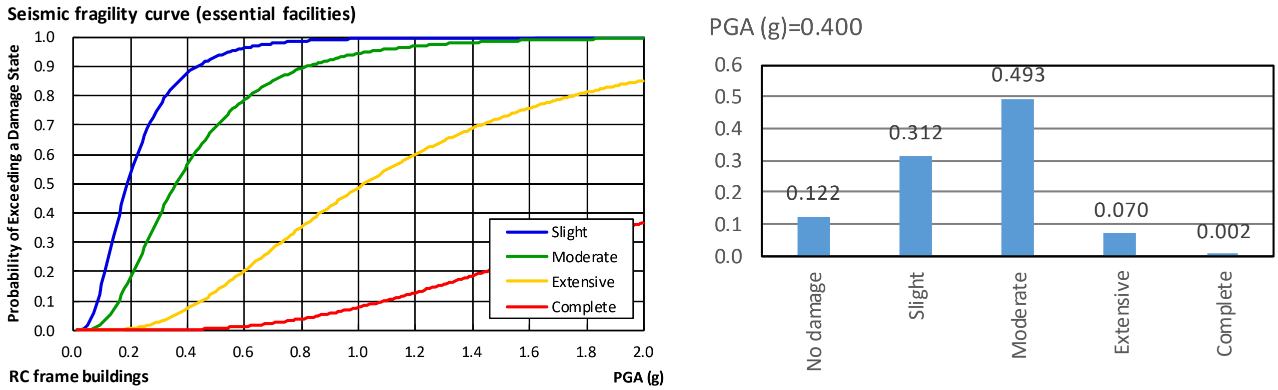


Seismic fragility functions



Probability mass function at PGA=0.4g

Figure 3-14. Seismic fragility functions & probability mass function at PGA=0.4g, Existing RC frame buildings



Seismic fragility functions — Probability mass function at PGA=0.4g

Figure 3-15. Seismic fragility functions & probability mass function at PGA=0.4g, Retrofitted RC frame buildings

3.6.1.4 Steel frame buildings

For steel frame buildings, the description in Hazus for the structural type, four damage states and fragility functions of existing and retrofitted buildings are presented as follows, and an expected degree of retrofit improvement is discussed.

3.6.1.4.1 Structural type for Steel frame

Steel frame (S1): These buildings have a frame of steel columns and beams. In some cases, the beam-column connections have very small moment resisting capacity but, in other cases, some of the beams and columns are fully developed as moment frames to resist lateral forces. Usually the structure is concealed on the outside by exterior nonstructural walls, which can be of almost any material (curtain walls, brick masonry, or precast concrete panels), and on the inside by ceilings and column furring. Diaphragms transfer lateral loads to moment-resisting frames. The diaphragms can be almost any material. The frames develop their stiffness by full or partial moment connections. The frames can be located almost anywhere in the building. Usually the columns have their strong directions oriented so that some columns act primarily in one direction while the others act in the other direction. Steel moment frame buildings are typically more flexible than shear wall buildings. This low stiffness can result in large inter-story drifts that may lead to relatively greater nonstructural damage.

3.6.1.4.2 Structural damage states description for Steel frame

FEMA Hazus (FEMA, 2003a) defines a number of damage states. The damage states for steel frame buildings are summarized in Table 3-16.

Damage state	State	Description
DS1	None	No observable damage
DS2	Slight	Minor deformations in connections or hairline cracks in few welds.
DS3	Moderate	Some steel members have yielded exhibiting observable permanent rotations at connections; few welded connections may exhibit major cracks through welds, or few bolted connections may exhibit broken bolts or enlarged bolt holes
DS4	Extensive	Most steel members have exceeded their yield capacity, resulting in significant permanent lateral deformation of the structure. Some of the structural members or connections may have exceeded their ultimate capacity exhibited by major permanent member rotations at connections, buckled flanges and failed connections. Partial collapse of portions of structure is possible due to failed critical elements and/or connections.

Damage state	State	Description
DS5	Complete	Significant portion of the structural elements have exceeded their ultimate capacities, or some critical structural elements or connections have failed, resulting in dangerous permanent lateral displacement, partial collapse or collapse of the building. Approximately 8% (low-rise), 5% (mid-rise) or 3% (high-rise) of the total area of S1 buildings with complete damage is expected to be collapsed.

Table 3-16. Steel frame building damage states (adapted from FEMA, 2003a)

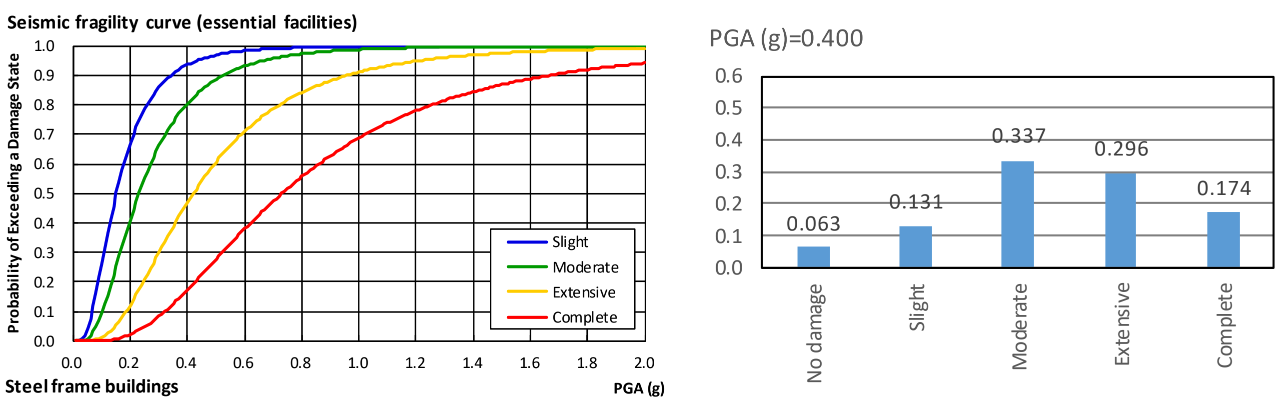
3.6.1.4.3 Fragility functions for Steel frame

In Table 3-17, the parameters of lognormal distribution for the fragility curves of existing and retrofitted steel frame buildings are listed. Based on these parameters, the fragility curves and relating probability mass function for existing and retrofitted steel frame buildings are presented in Figure 3-16 and Figure 3-17.

As shown in the fragility functions, the curves of all damage states are largely improved by seismic retrofit implementation. For an earthquake with a PGA of 0.4 g (an average value expected from an earthquake occurring in the Caribbean once in 475 years), the probability of "No damage" is 6% for the existing building and 9% for the retrofitted building; the probability of exceeding "Extensive damage" is 47% for the existing building and 12% for the retrofitted building. Thus, seismic retrofit can be expected to greatly reduce the probability of seismic damage to steel frame buildings.

Type	Status	Median (g); PGA				Lognormal standard deviation (g)
		Slight	Moderate	Extensive	Complete	
RC frame	Existing	0.15	0.23	0.42	0.73	0.64
	Retrofitted	0.17	0.34	0.85	2.10	0.64

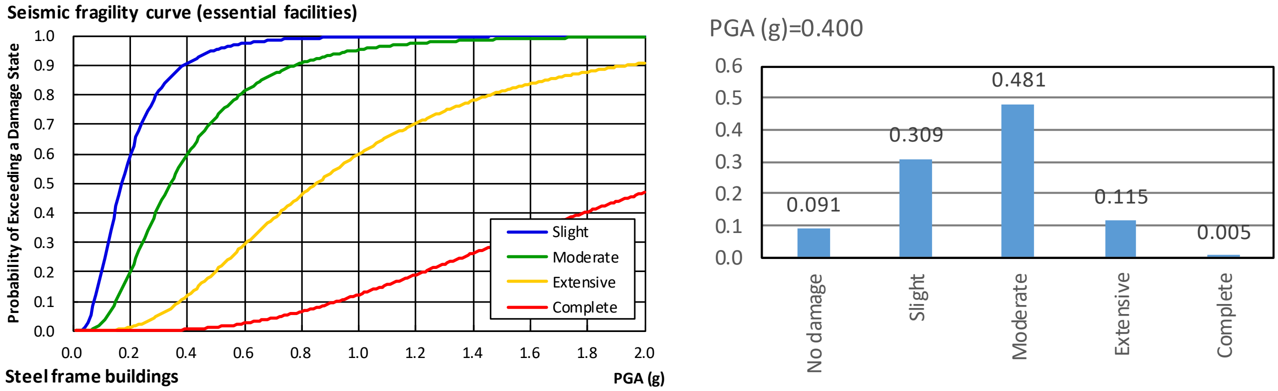
Table 3-17. Parameters of seismic fragility functions for steel frame buildings (FEMA, 2013a)



Seismic fragility functions

Probability mass function at PGA=0.4g

Figure 3-16. Seismic fragility functions & probability mass function at PGA=0.4g, Existing steel frame buildings



Seismic fragility functions　　　　　　　　Probability mass function at PGA=0.4g

Figure 3-17. Seismic fragility functions & probability mass function at PGA=0.4g, Retrofitted steel frame buildings

3.6.1.5 Steel frame high-rise buildings

For steel frame high-rise buildings, the description in Hazus for the structural type, four damage states and fragility functions of existing and retrofitted buildings are presented as follows, and an expected degree of retrofit improvement is discussed.

3.6.1.5.1 Structural type for Steel frame high-rise

> *Steel frame (S1): These buildings have a frame of steel columns and beams. In some cases, the beam-column connections have very small moment resisting capacity but, in other cases, some of the beams and columns are fully developed as moment frames to resist lateral forces. Usually the structure is concealed on the outside by exterior nonstructural walls, which can be of almost any material (curtain walls, brick masonry, or precast concrete panels), and on the inside by ceilings and column furring. Diaphragms transfer lateral loads to moment-resisting frames. The diaphragms can be almost any material. The frames develop their stiffness by full or partial moment connections. The frames can be located almost anywhere in the building. Usually the columns have their strong directions oriented so that some columns act primarily in one direction while the others act in the other direction. Steel moment frame buildings are typically more flexible than shear wall buildings. This low stiffness can result in large inter-story drifts that may lead to relatively greater nonstructural damage.*

3.6.1.5.2 Structural damage states description for Steel frame high-rise

FEMA Hazus (FEMA, 2003a) defines a number of damage states. The damage states for steel frame high-rise buildings are summarized in Table 3-18.

Damage state	State	Description
DS1	None	No observable damage
DS2	Slight	Minor deformations in connections or hairline cracks in few welds.
DS3	Moderate	Some steel members have yielded exhibiting observable permanent rotations at connections; few welded connections may exhibit major cracks through welds, or few bolted connections may exhibit broken bolts or enlarged bolt holes.
DS4	Extensive	Most steel members have exceeded their yield capacity, resulting in significant permanent lateral deformation of the structure. Some of the structural members or connections may have exceeded their ultimate capacity exhibited by major permanent member rotations at connections, buckled flanges and failed connections. Partial collapse of portions of structure is possible due to failed critical elements and/or connections.

Damage state	State	Description
DS5	Complete	Significant portion of the structural elements have exceeded their ultimate capacities, or some critical structural elements or connections have failed, resulting in dangerous permanent lateral displacement, partial collapse or collapse of the building. Approximately 8% (low-rise), 5% (mid-rise) or 3% (high-rise) of the total area of S1 buildings with complete damage is expected to be collapsed.

Table 3-18. Steel frame high rise building damage states (adapted from FEMA, 2003a)

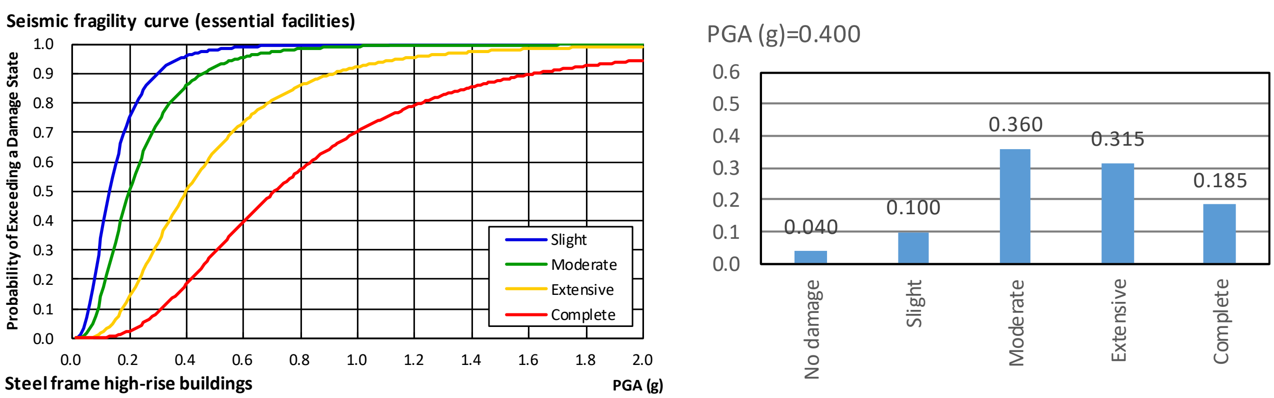
3.6.1.5.3 Fragility functions for Steel frame high-rise

In Table 3-19, the parameters of lognormal distribution for the fragility curves of existing and retrofitted steel frame high-rise buildings are listed. Based on these parameters, the fragility curves and relating probability mass function for existing and retrofitted steel frame high-rise buildings are presented in Figure 3-18 and Figure 3-19.

As shown in the fragility functions, the curves of all damage states are largely improved by seismic retrofit implementation. For an earthquake with a PGA of 0.4 g (an average value expected from an earthquake occurring in the Caribbean once in 475 years), the probability of "No damage" is 4% for the existing building and 4% for the retrofitted building; the probability of exceeding "Extensive damage" is 50% for the existing building and 22% for the retrofitted building. Thus, seismic retrofit can be expected to greatly reduce the probability of severe seismic damage to steel frame high-rise buildings.

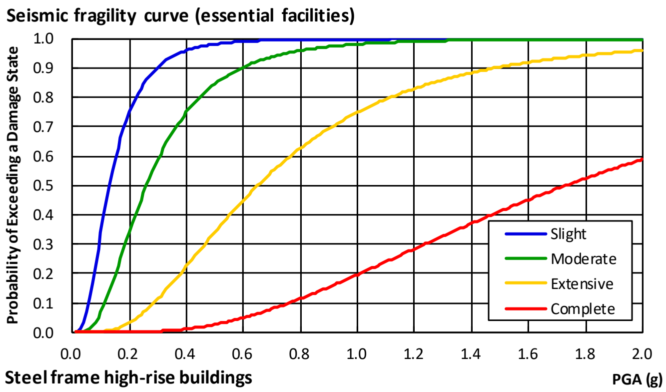
Type	Status	Median (g); PGA				Lognormal standard deviation (g)
		Slight	Moderate	Extensive	Complete	
RC frame	Existing	0.13	0.20	0.40	0.71	0.64
	Retrofitted	0.13	0.26	0.65	1.73	0.64

Table 3-19. Parameters of seismic fragility functions for steel frame high-rise buildings (FEMA, 2013a)

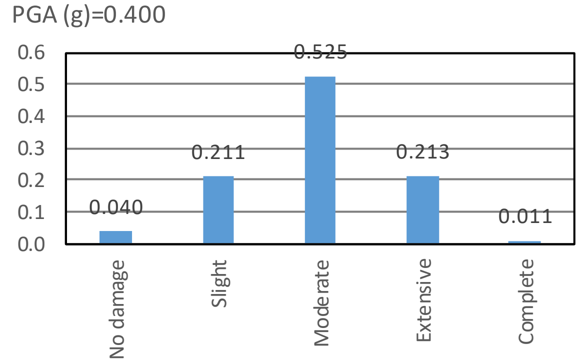


Seismic fragility functions Probability mass function at PGA=0.4g

Figure 3-18. Seismic fragility functions & probability mass function at PGA=0.4g, Existing steel frame high-rise buildings



Seismic fragility functions



Probability mass function at PGA=0.4g

Figure 3-19. Seismic fragility functions & probability mass function at PGA=0.4g, Retrofitted steel frame high-rise buildings

3.6.2 Earthquake liquefaction: FEMA Hazus fragility functions

3.6.2.1 Overview

FEMA Hazus (FEMA, 2013a) developed a fragility function for earthquake liquefaction causing potential ground failure at a building site. Permanent ground displacements (i.e., lateral spreading and ground settlement) due to liquefaction are applied for the intensity index of the fragility function, and one damage state (i.e., weighed average of Extensive and Complete structural damage, see 3.6.1 for damage state definitions according to structural type) is arranged to express liquefaction fragility. Here, the liquefaction damage probability presented in Hazus is based on a condition under which liquefaction occurs during an earthquake. Since earthquake liquefaction is a type of ground hazard, this fragility curve can be assumed to apply to all types of buildings built on shallow or unknown foundations. The estimation process, liquefaction fragility functions and improvement consequence are summarized as follows.

3.6.2.2 Estimation process for a liquefaction damage probability

1) Based on the soil characteristics and expected earthquake intensity at the site, calculate the permanent ground displacement of lateral spreading and ground settlement.
2) Apply the permanent ground displacements to each fragility function and identify the liquefaction damage probability.
3) Take the larger probability as a liquefaction damage probability and combine with a damage ratio of Extensive/Complete damage state of building.

3.6.2.3 Liquefaction fragility functions (lateral spreading and ground settlement)

As well as the earthquake shaking fragility functions for buildings, the liquefaction fragility function is expressed by lognormal distribution with permanent ground displacement. The parameters of the functions are listed in Table 3-20 and the fragility functions are depicted in Figure 3-20.

Failure mode	**Median (in.); PGD**	**Lognormal standard deviation (in.)**
	Extensive/Complete	
Lateral spreading	10	1.2
Ground settlement	60	1.2

Table 3-20. Parameters of liquefaction fragility functions (FEMA, 2013a)

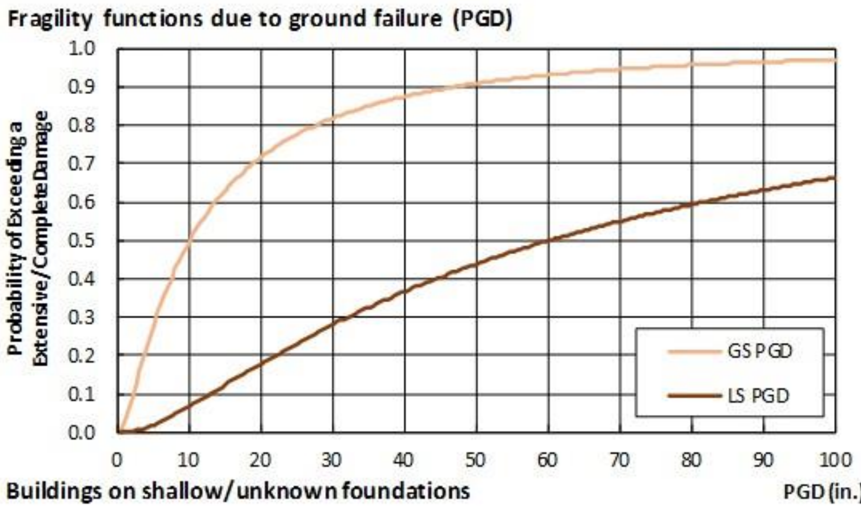


Figure 3-20. Liquefaction fragility functions for buildings with shallow/unknown foundations

3.6.2.4 Liquefaction damage after retrofit implementation

Since liquefaction improvement is usually a substantial measure, the liquefaction hazard is, for buildings in this study, assumed to be essentially eliminated once such a measure is implemented.

3.6.3 Wind: FEMA Hazus fragility functions

3.6.3.1 Overview

The roof, walls and the connections of buildings are susceptible to damage from wind load caused mainly by peak gust wind speed (PGWS) due to hurricane. For the fragility functions for wind hazards like hurricanes, FEMA Hazus (FEMA, 2013b) provides a number of wind fragility functions associated with wind speed for buildings according to the materials and connections of structural components (like the roof and wall). Two sets of fragility functions are basically developed by Hazus: one for buildings with low capacity of roof-wall connection and one for buildings in which the strength of roof-wall connections is more robust. In this study, it is considered that the former fragility function is suitable for representing the existing building and the latter one is proper for the retrofitted building. Then, the difference between these two fragility functions is assumed to express the improvement extent of structural strengthening against wind hazard. The fragility parameters of these functions are adapted to estimate the damage probability of buildings for wind hazard and the damage probabilities estimated by fragility functions are used for the performance comparison between existing and retrofitted buildings. In the following subsections, the wind fragility functions are described for the representative structural types of buildings (i.e., masonry/concrete buildings as heavy structures and wood/steel buildings for light structures).

3.6.3.2 Masonry/Concrete buildings

Two sets of wind fragility functions are selected for masonry/concrete buildings. Each fragility function (existing or retrofitted) consists of five damage states. The fragility functions and the description of damage states prepared in Hazus are presented below, and an expected degree of retrofit improvement is discussed.

3.6.3.2.1 Structural damage states description for Masonry/Concrete buildings

The damage descriptions and typical damage amount for specific components are shown in Table 3-21, according to each damage state.

Damage State	Damage description	Roof cover failure	Window/ Door failure	Roof deck failure	Missile impacts on walls	Roof structure failure	Wall structur e failure
No damage/	Little or no visible damage from the outside. No broken	≤ 2%	No	No	No	No	No

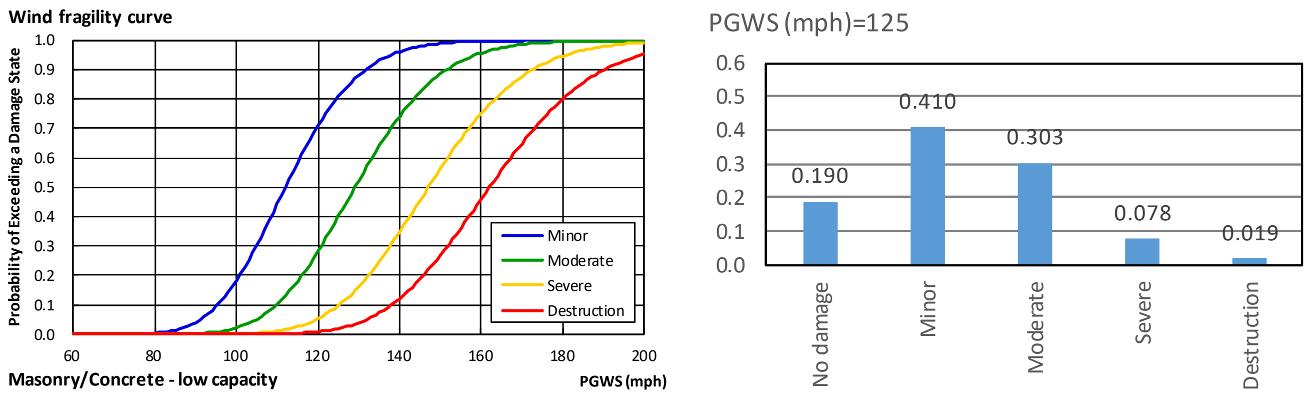
very minor damage	windows, or failed roof deck. Minimal loss of roof over, with no or very limited water penetration.						
Minor damage	Maximum of one broken window, door or garage door. Moderate roof cover loss that can be covered to prevent additional water entering the building. Marks or dents on walls requiring No painting or patching for repair.	> 2% and ≤ 15%	One window, door, or garage door failure	No	< 5 impacts	No	No
Moderate damage	Major roof cover damage, moderate window breakage. Minor roof sheathing failure. Some resulting damage to interior of building from water.	> 15% and ≤ 50%	> one and ≤ the larger of 20% & 3	1 to 3 panels	Typically 5 to 10 impacts	No	No
Severe damage	Major window damage or roof sheathing loss. Major roof cover loss. Extensive damage to interior from water.	> 50%	> the larger of 20% & 3 and ≤ 50%	> 3% and ≤ 25%	Typically 10 to 20 impacts	No	No
Destruction	Complete roof failure and/or, failure of wall frame. Loss of more than 50% of roof sheathing.	Typically > 50%	> 50%	> 25%	Typically > 20 impacts	Yes	Yes

Table 3-21. Damage states for Masonry/Concrete buildings (FEMA, 2013b)

3.6.3.2.2 Fragility functions for Masonry/Concrete buildings

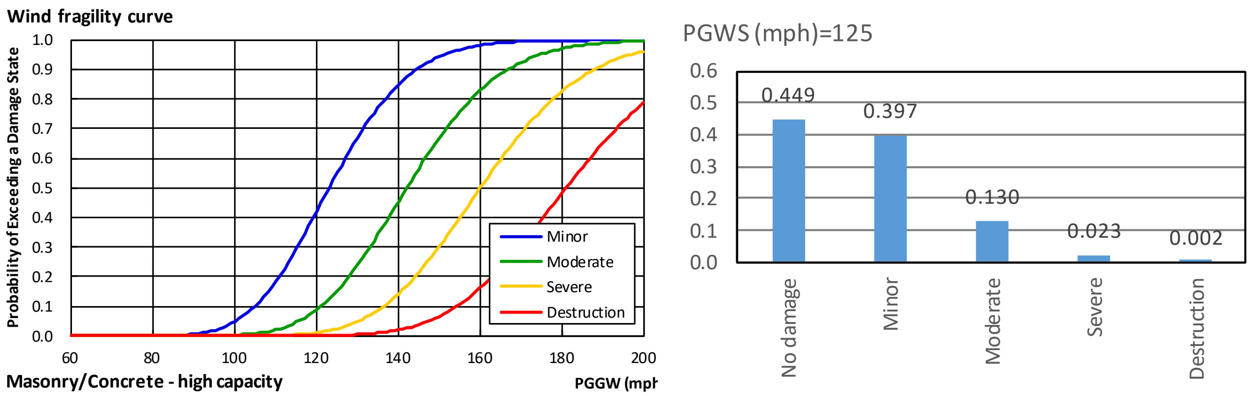
The parameters of lognormal distribution for wind fragility functions were read from the original fragility functions developed by Hazus (FEMA, 2013b). Based on the parameters, the wind fragility functions and relating probability mass function for existing and retrofitted masonry/concrete buildings are presented in Figure 3-21 and Figure 3-22.

As shown in the fragility functions, the curves of all damage states are largely improved by wind retrofit implementation. For a strong wind with a Peak Gust Wind Speed (PGWS) of 125 mph (a typical wind speed for a major Category 3 hurricane), the probability of “No damage” is 19% for an existing building and 45% for a retrofitted building; the probability of exceeding “Severe damage” is 10% for an existing building and 3% for a retrofitted building. Thus, wind retrofit can be expected to greatly reduce the probability of wind damage to masonry/concrete buildings.



Wind fragility functions
Probability mass function at PGWS=125 mph

Figure 3-21. Wind fragility functions & probability mass function at PGWS=125 mph, Existing Masonry / Concrete buildings



Wind fragility functions
Probability mass function at PGWS=125 mph

Figure 3-22. Wind fragility functions & probability mass function at PGWS=125 mph, Retrofitted Masonry / Concrete buildings

3.6.3.3 Wood/Steel buildings

Two sets of wind fragility functions are selected for wood/steel buildings. Each fragility function (existing or retrofitted) consists of five damage states. The fragility functions and the description of damage states prepared in Hazus are presented below, and an expected degree of retrofit improvement is discussed.

3.6.3.3.1 Structural damage states description for Wood/Steel buildings

The damage descriptions and typical damage amount for specific components are shown in Table 3-22, according to each damage state.

Damage state	Damage description	Roof cover failure	Window/ Door failure	Roof deck failure	Missile impacts on walls	Joist failure
No damage/very minor damage	Little or no visible damage from the outside. No broken windows, or failed roof deck. Minimal loss of roof cover, with no or very limited water penetration.	≤ 2%	No	No	No	No
Minor damage	Maximum of one broken window or door. Moderate roof cover loss that can be covered to prevent additional water	> 2% to ≤ 15%	One window or door	No	Typically < 5 impacts	No

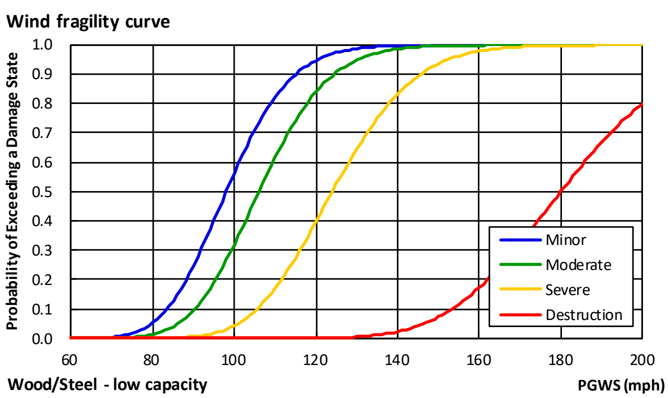
	entering the building. Marks or dents on walls requiring painting or patching for repair.					
Moderate damage	Major roof cover damage, moderate window breakage. Minor roof deck failure. Some resulting damage to interior of building from water.	> 15% to ≤ 50%	> One to ≤ 2%	One or two panels	Typically 5 to 10 impacts	No
Severe damage	Major window damage or roof sheathing loss. Major roof cover loss. Extensive damage to interior from water. Limited, local joist failures.	> 50%	> 2% to ≤ 25%	> Two to ≤ 25%	Typically 10 to 20 impacts	One joist to ≤ 25%
Destruction	Essentially complete roof failure and/or of more than 25% of roof sheathing. Significant amount of the wall envelope opened through windows failure. Extensive damage to interior.	Typically > 50%	> 25%	> 25%	Typically > 20 impacts	> 25%

Table 3-22. Damage states for wood/steel buildings (FEMA, 2013b)

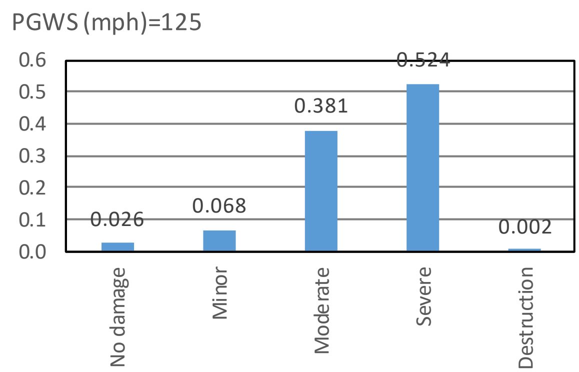
3.6.3.3.2 Fragility functions for Wood/Steel buildings

The parameters of lognormal distribution for wind fragility functions are read from the original fragility functions developed by Hazus (FEMA, 2013b). Based on the parameters, the wind fragility functions and relating probability mass function for existing and retrofitted wood/steel buildings are presented in Figure 3-23 and Figure 3-24.

As shown in the fragility functions, the curves of all damage states are largely improved by wind retrofit implementation. For a strong wind with a PGWS of 125 mph (a typical wind speed for a major Category 3 hurricane), the probability of “No damage” is 3% for an existing building and 6% for a retrofitted building; the probability of exceeding “Severe damage” is 53% for an existing building and 29% for a retrofitted building. Thus, wind retrofit can be expected to greatly reduce the probability of wind damage to wood/steel buildings.

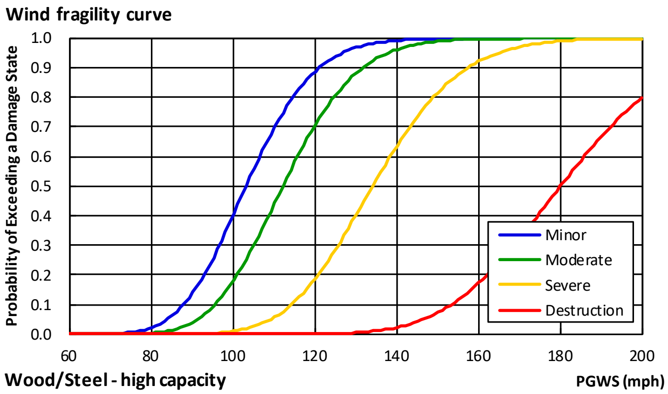


Wind fragility functions

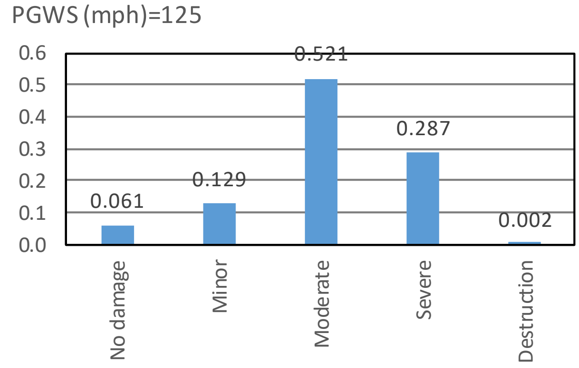


Probability mass function at PGWS=125 mph

Figure 3-23. Wind fragility functions & probability mass function at PGWS=125 mph, Existing Wood/Steel buildings



Wind fragility functions



Probability mass function at PGWS=125 mph

Figure 3-24. Wind fragility functions & probability mass function at PGWS=125 mph, Retrofitted Wood/Steel buildings

3.6.4 Flood: FEMA Hazus damage functions

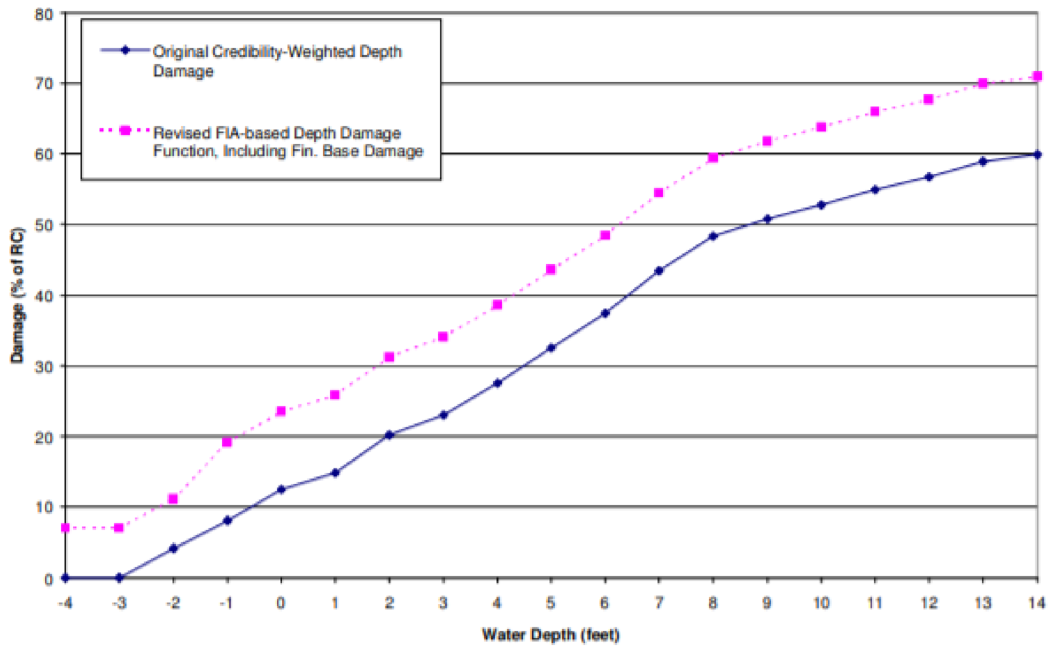
FEMA Hazus (FEMA, 2013c) developed two types of damage functions for flood hazard. One of the functions measures building damage amount (i.e., percent of replacement cost) corresponding to flood inundation depth, and another one scales the collapse potential of the building based on a combination of water velocity and depth. As seen in the damage functions, the depth of flood inundation is a key factor. Hazus also mentions in the document that (FEMA, 2013c):

> *In general, it is expected that the major structural components of a building will survive a flood, but that the structural finishes and contents/inventory may be severely damaged due to inundation.*

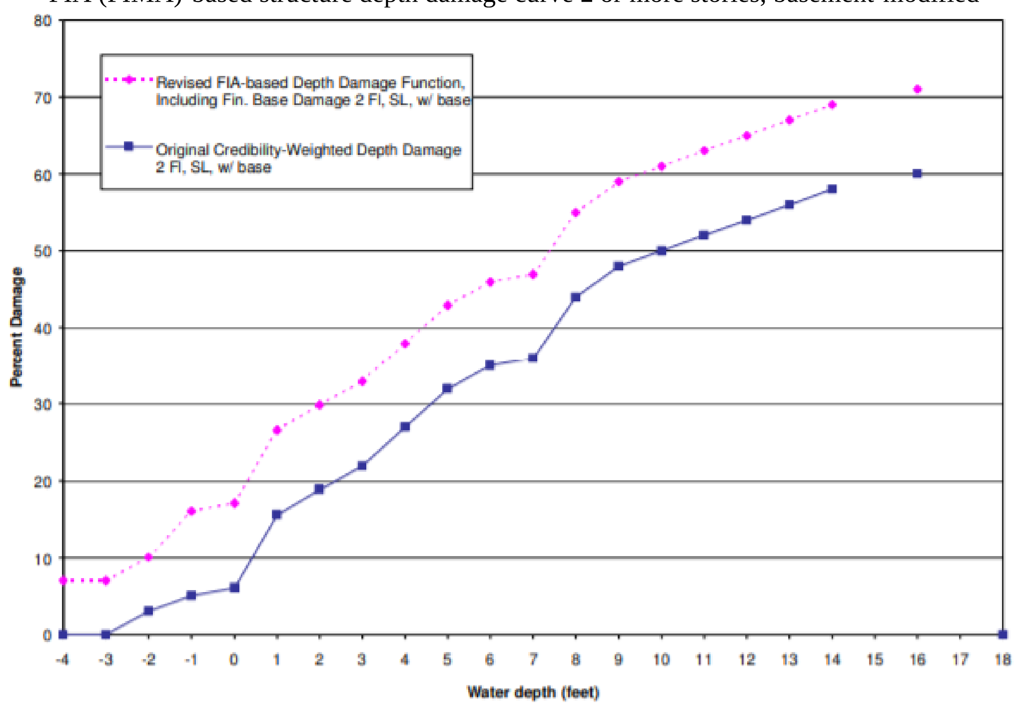
The damage functions and improvement consequence are summarized as follows.

3.6.4.1 Depth-damage curve for flood hazard

The depth damage curves presented in Hazus are compiled from various loss data recorded by the Federal Insurance and Mitigation Administration (FIMA), the U.S. Army Corps of Engineers (USACE) and the USACE Institute for Water Resources (USACE IWR). Example functions are shown in Figure 3-25; the damage percentage can be identified by the inundation depth and is approximately 35% damage occurring at 3 feet inundation.



FIA (FIMA)-based structure depth damage curve 2 or more stories, basement-modified

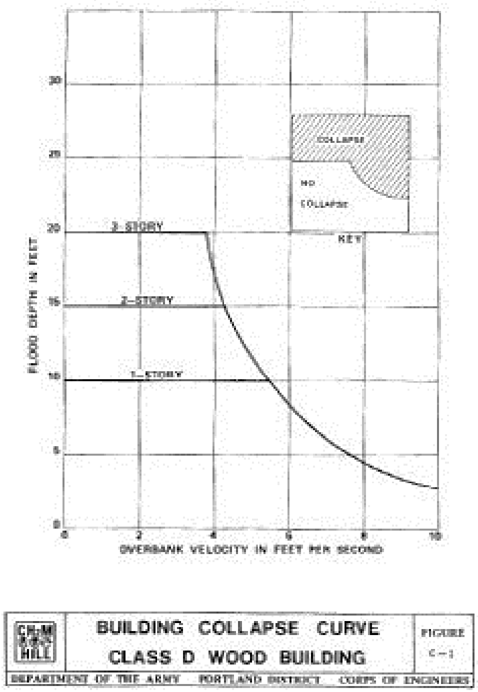


FIA (FIMA)-based structure depth damage curve Split level, basement-modified

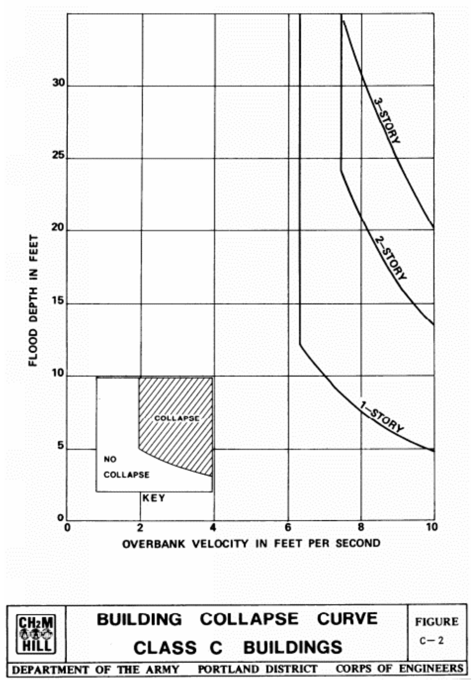
Figure 3-25. Building depth damage curves for buildings (FEMA, 2013c)

3.6.4.2 Velocity depth-damage curve for flood hazard

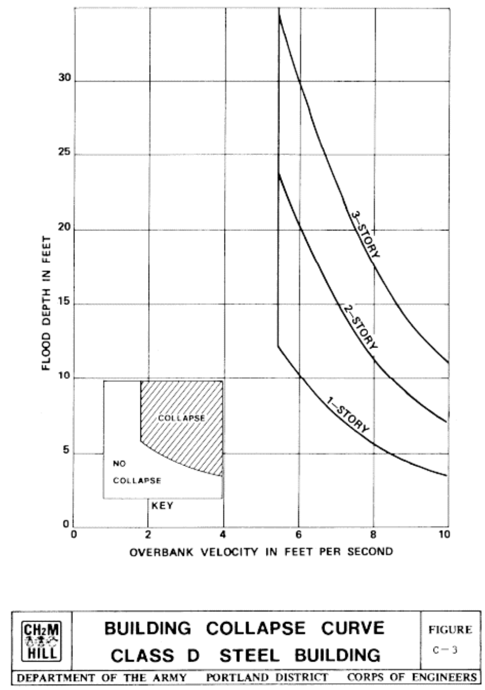
Flood with high velocity combined with water inundation could result in not only content loss, but also structural damage. For example, large water pressure due to flooding might separate buildings from their foundations if the connections to the foundations do not possess enough capacity for the water pressure. Hazus adopted the velocity-based building collapse functions developed by the Portland District of the U.S. Army Corp of Engineers, as shown in Figure 3-26. According to the structural type and height of the building, the collapse potential (i.e., collapse or not collapse) due to flooding is expressed by a combination of inundation depth and overbank water velocity. As seen in the functions, the collapse potential is very low for buildings at a certain low depth of inundation.



Wood building



Masonry/Concrete building



Steel building

Figure 3-26. Building velocity-depth damage curves for various structural types (FEMA, 2013c)

3.6.4.3 Flood damage after retrofit implementation

Since flood improvement is typically a substantial measure to protect buildings from water invasion, the damage due to flood hazard is assumed to be essentially eliminated once such a measure is implemented in this study.

3.7 Strengthening techniques

This section shows general strengthening measures for the vulnerable components of buildings depending on the type of hazard considered in this study. As example retrofit techniques, measures implemented in modern building-code countries like the U.S. or Japan are focused. The subsequent section provides cost information for strengthening measures where specific cost data for the Caribbean is available.

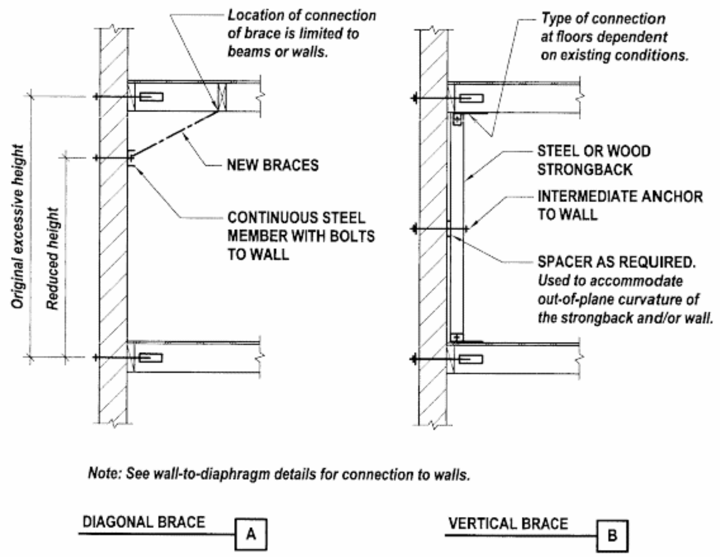
3.7.1 Earthquake: ground shaking

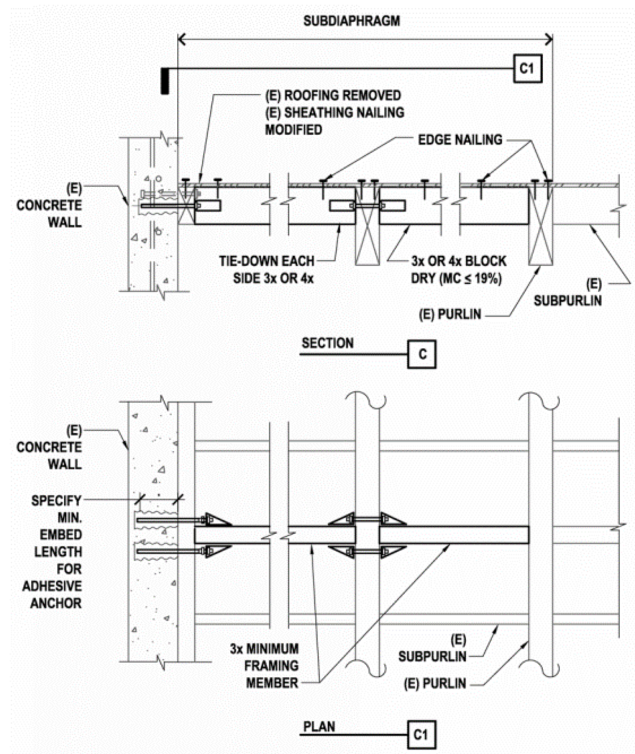
For earthquake shaking hazard, strengthening the in-plane capacity and stiffness and improving structural irregularity and ductility are common measures to increase building resiliency. Improving the out-of-plane resistance of walls is effective to reduce severe damage to old or non-engineered buildings, especially URM buildings. In addition, installing an energy dissipation system could be a retrofit option for the most important buildings to be accessed in post-disaster circumstances (e.g., emergency center or medical facility).

3.7.1.1 Wall out-of-plane strengthening

Old or unreinforced buildings (e.g., unreinforced masonry buildings) typically do not possess appropriate reinforcement detailing required by a modern building code. Thus, the out-of-plane bending resistance of the buildings is usually inadequate. The failure of heavy walls due to a lack of out-of-plane resistance is historically the most devastating damage to buildings and people living in or walking by the building.

Bracing the wall at regularly-spaced intervals is a general measure for improving the resistance of out-of-plane failure. To brace the wall, a steel strongback, existing diaphragm (i.e., floor element), concrete pilaster and post-installed anchor are applied; see Figure 3-27 and Figure 3-28. This improvement can be installed on both the inside and outside of the building, but the inside approach is more popular for heritage building preservation purposes or architectural exterior finishes.

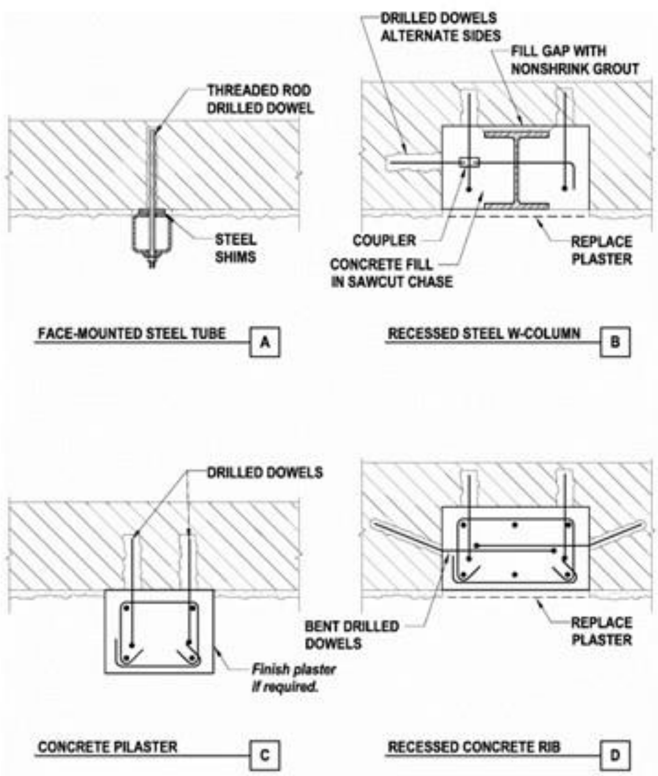




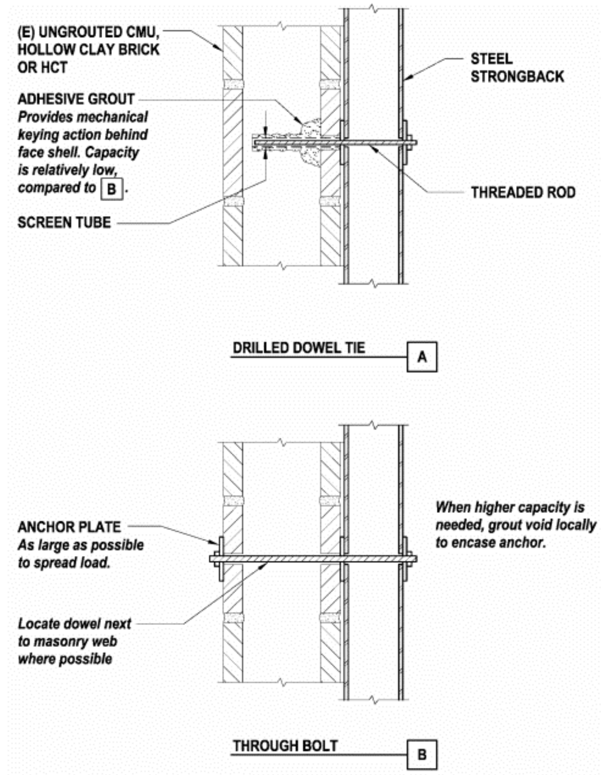
Out-of-plane wall bracing

Anchorage from diaphragm to wall

Figure 3-27. Out-of-plane strengthening for masonry wall (FEMA, 2006b)



Vertical element connections



Strongback connections

Figure 3-28. Vertical element connections of out-of-plane strengthening (FEMA, 2006b)

3.7.1.2 In-plane strengthening

The global seismic strength of a building heavily relies on the in-plane strength of vertical elements, such as shear walls, braced frames or moment frames. These elements are essential components constituting the structural integrity and seismic robustness of a building. A lack of in-plane seismic capacity could therefore cause significant damage that requires extensive repair work or a complete collapse of the building that necessitates demolition and replacement.

To improve the in-plane seismic strength of a building, a new element, such as a shear wall or braced frame, is typically added to the existing structural system, as shown in Figure 3-29 and Figure 3-30. Adding new elements from the outside is more convenient for building usability and retrofit constructability, but inside installation is also constructible with a careful installation plan. Typical configurations and details of reinforced concrete (RC) shear walls and steel buckling restrained braces (BRB) are shown in Figure 3-31 and Figure 3-32, respectively. In both cases, it is important to provide the appropriate load path elements to transfer the seismic force from the new elements through new foundations to the ground.

Enhancing the existing in-plane element is also an effective measure. For an existing concrete wall, a methodology to attach fiber-reinforced polymer (FRP) sheets was developed to strengthen the shear strength and improve the ductility of the wall. The attachment elevation and details of anchoring FRP to concrete walls are presented in Figure 3-33. Thickening the existing masonry shear wall is also an effectual way to increase the in-plane seismic strength. Concrete/shotcrete shear wall is usually overlaid on the existing masonry wall with the connecting dowels and new foundation as needed. Typical section details and elevation configuration of overlaid concrete walls are shown in Figure 3-34.

Figure 3-29. Typical in-plane strengthening, RC shear wall (FEMA, 2006a)

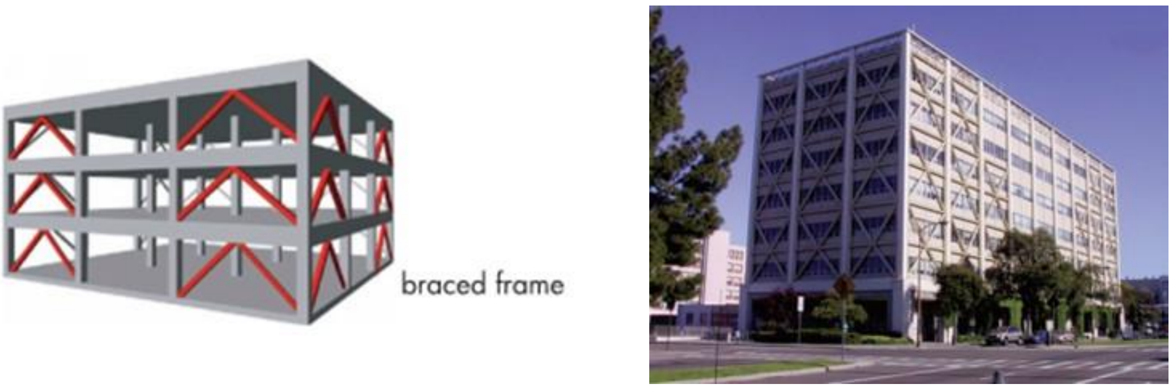
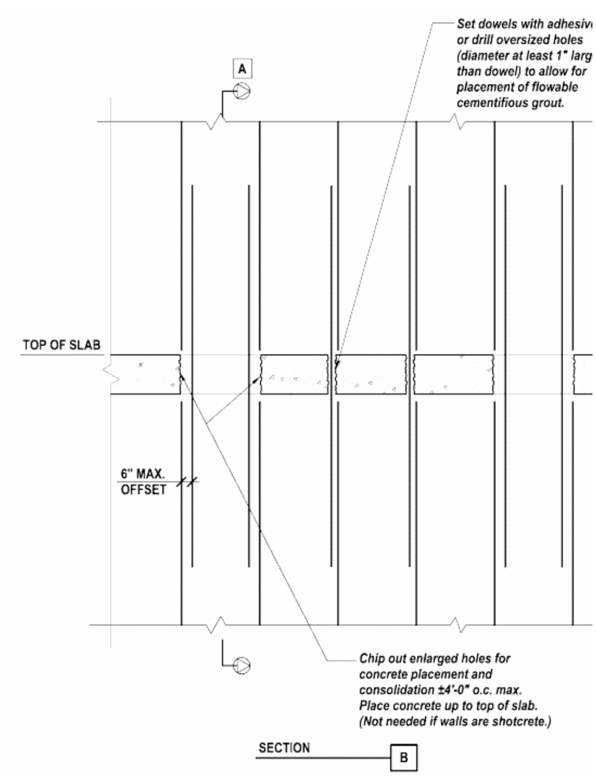
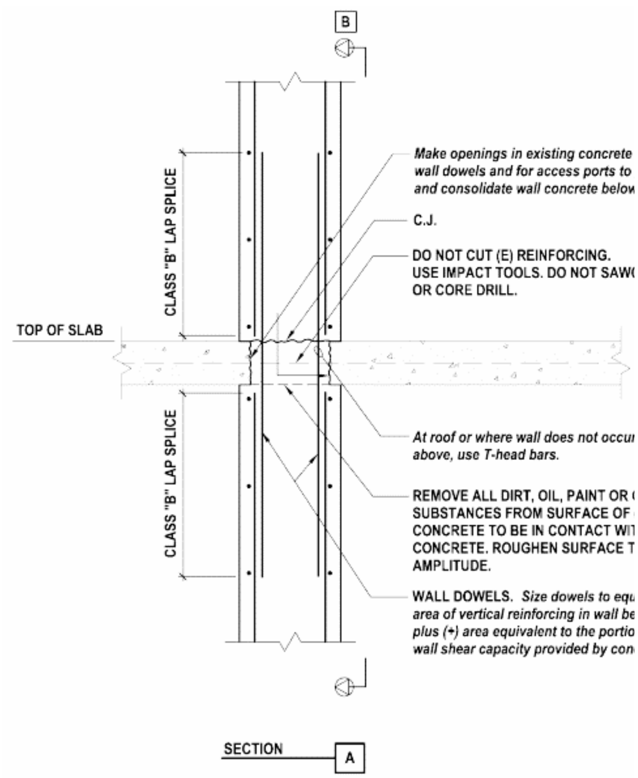


Figure 3-30. Typical in-plane strengthening, Steel brace (FEMA, 2006a)

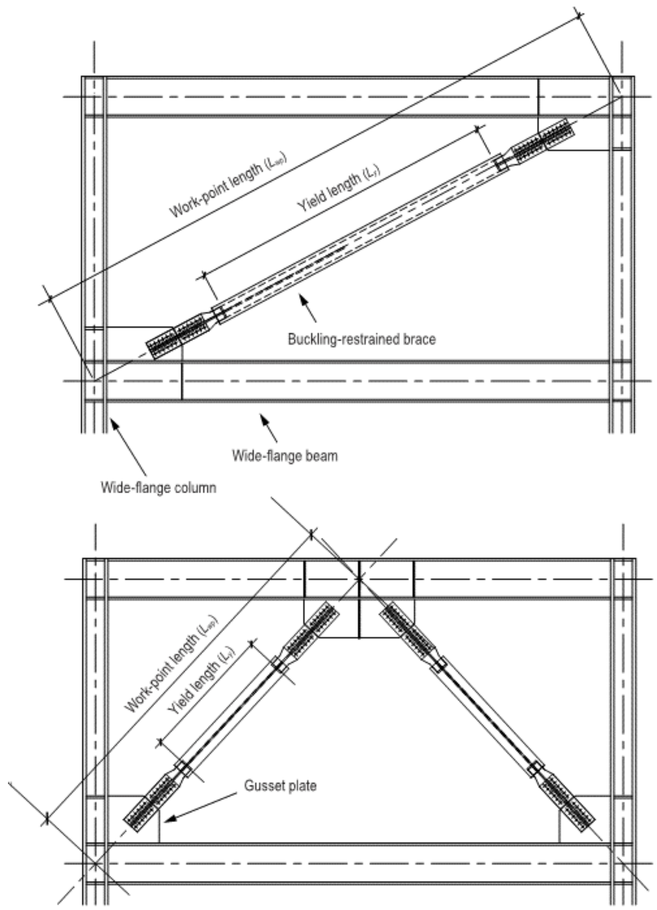


Wall elevation with existing slab

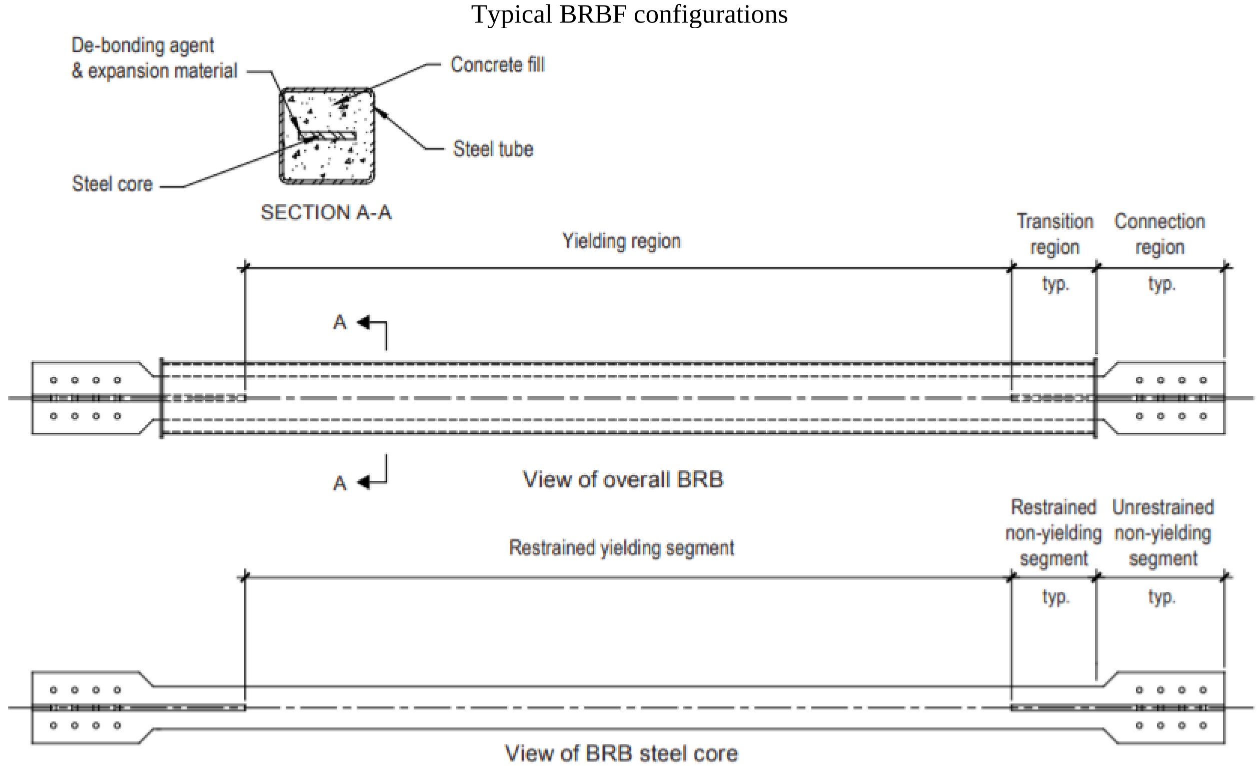


Wall connection with existing slab

Figure 3-31. RC shear wall strengthening (FEMA, 2006b)



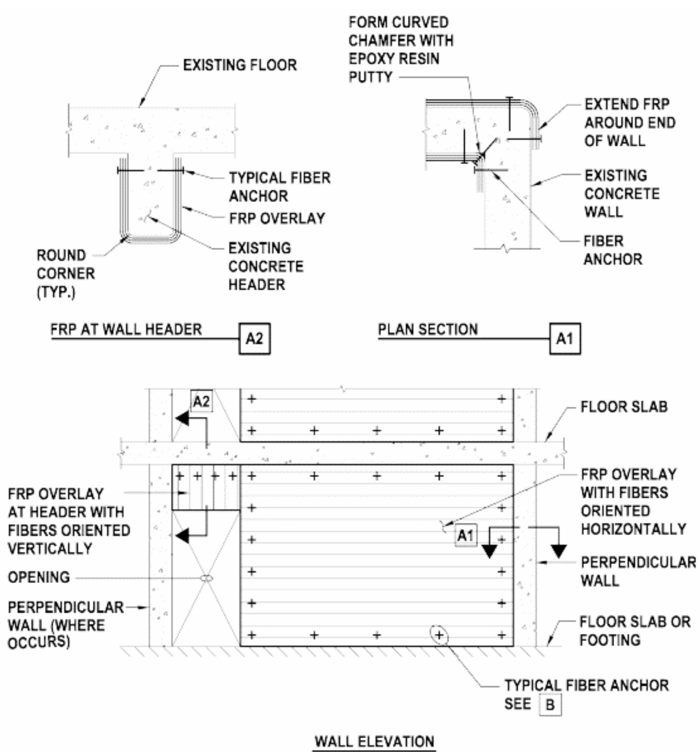
Typical BRBF configurations



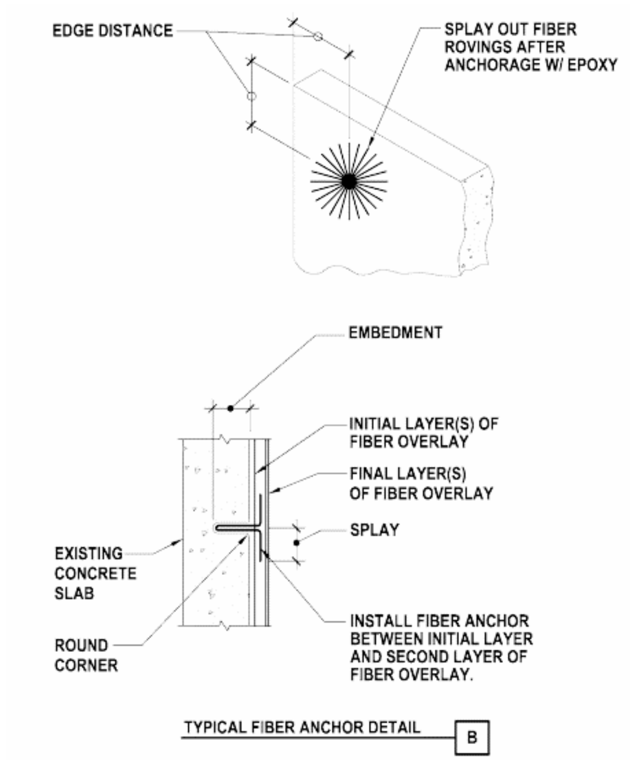
Common BRB assembly

BRBF example

Figure 3-32. Steel buckling restrained brace (BRB) strengthening (NIST, 2015)

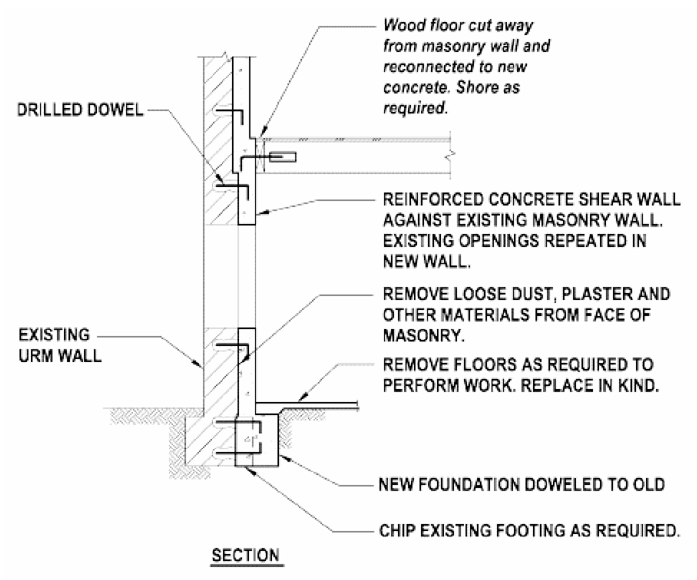


FRP shear strengthening on existing wall

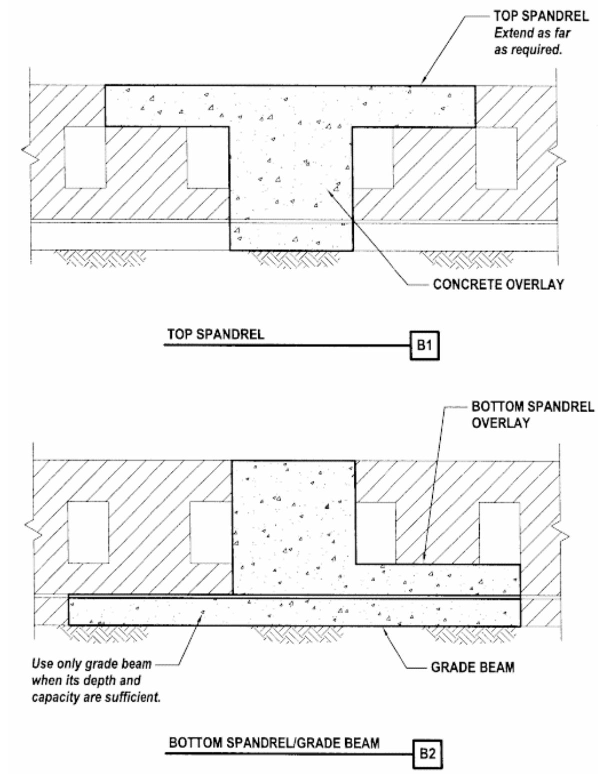


FRP anchor to existing wall

Figure 3-33. FRP strengthening on existing RC shear wall (FEMA, 2006b)



Concrete/shotcrete wall overlay



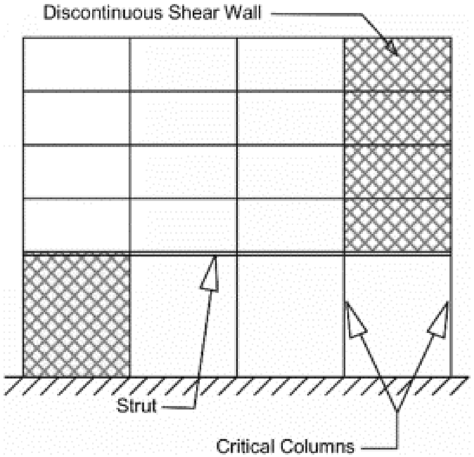
Wall addition elevation

Figure 3-34. RC shear wall addition (or shotcrete) on existing masonry wall (FEMA, 2006b)

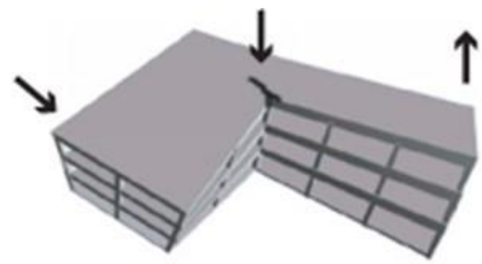
3.7.1.3 Structural irregularity improvement

Structural irregularity due to unbalanced configuration or strength/stiffness is one of the most important factors for the structural stiffness and strength of buildings; see Figure 3-35. For example, the vertical discontinuity or horizontal offset of a structural wall could cause serious damage to critical columns right below the upper floor wall or could induce large harmful displacement at weak locations due to torsional movement. The stress/force concentration is usually generated at the corner of irregular plan shapes and severe damage would happen on the elements around the corner. Weak/soft story (typically at the first floor) due to insufficient structural elements like walls or braces might bring a large permanent residual displacement or a pancake collapse (i.e., completely crush the story) on the floor. In that case, the whole building might have to be demolished or replaced because of the destructive floor damage, even though other floors may look intact.

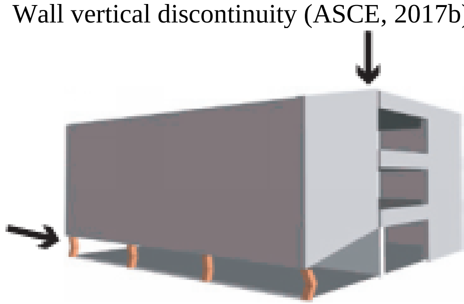
Common measures to reduce structural irregularities and improve the structural performance are adding shear walls, braced frames and moment frames, as shown in Figure 3-36. The in-plane strengthening measures presented in Section 3.7.1.2 can be applied. As can be seen in Figure 3-36, adding walls/braces to the weak story can usually diminish multiple structural irregularities. For example, the new walls/braces could eliminate the weak/soft story effect, reduce the torsional movement, make the seismic displacement smaller and decrease the stress concentration. This kind of retrofit for structural irregularities can help avoid severe global damage of buildings and hurtful effects on occupants and facilities.



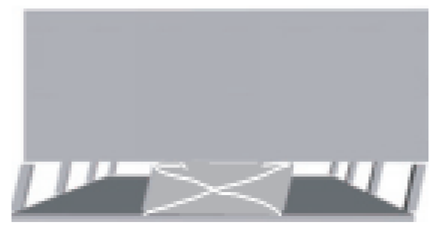
Wall vertical discontinuity (ASCE, 2017b)



Irregular plan; re-entrant corner (FEMA, 2006a)

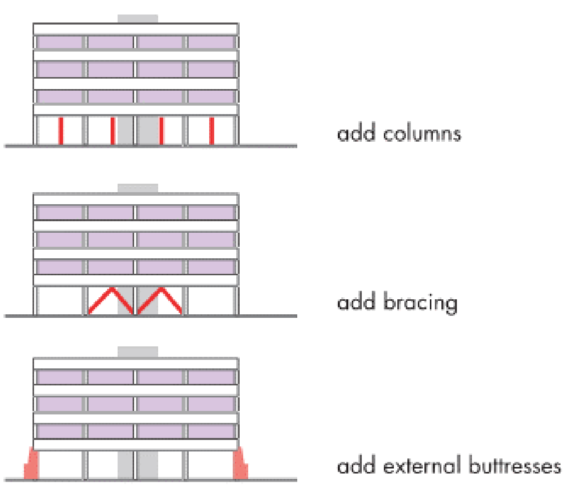


Irregular elevation; wall offset (FEMA, 2006a)

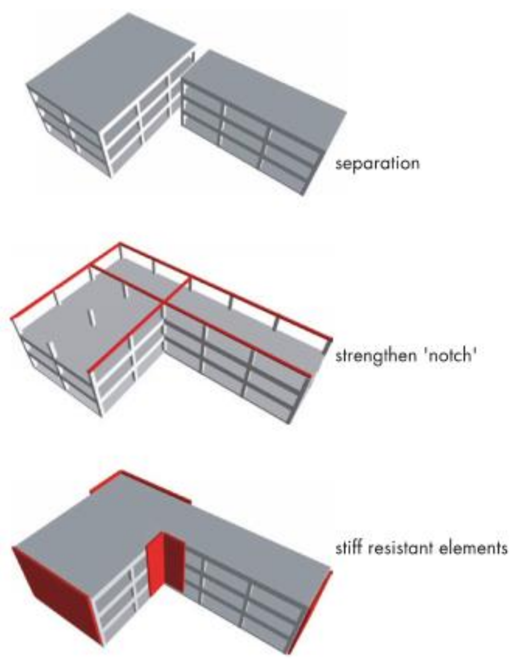


Soft/weak story (FEMA, 2006a)

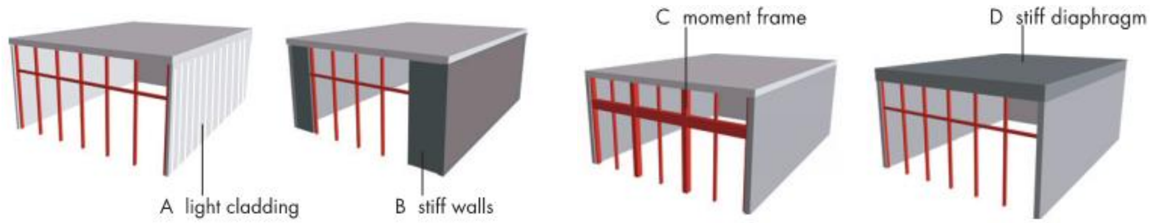
Figure 3-35. Examples of structural irregularities of buildings



Retrofit measures for soft first story



Strengthening solutions for re-entrant corner



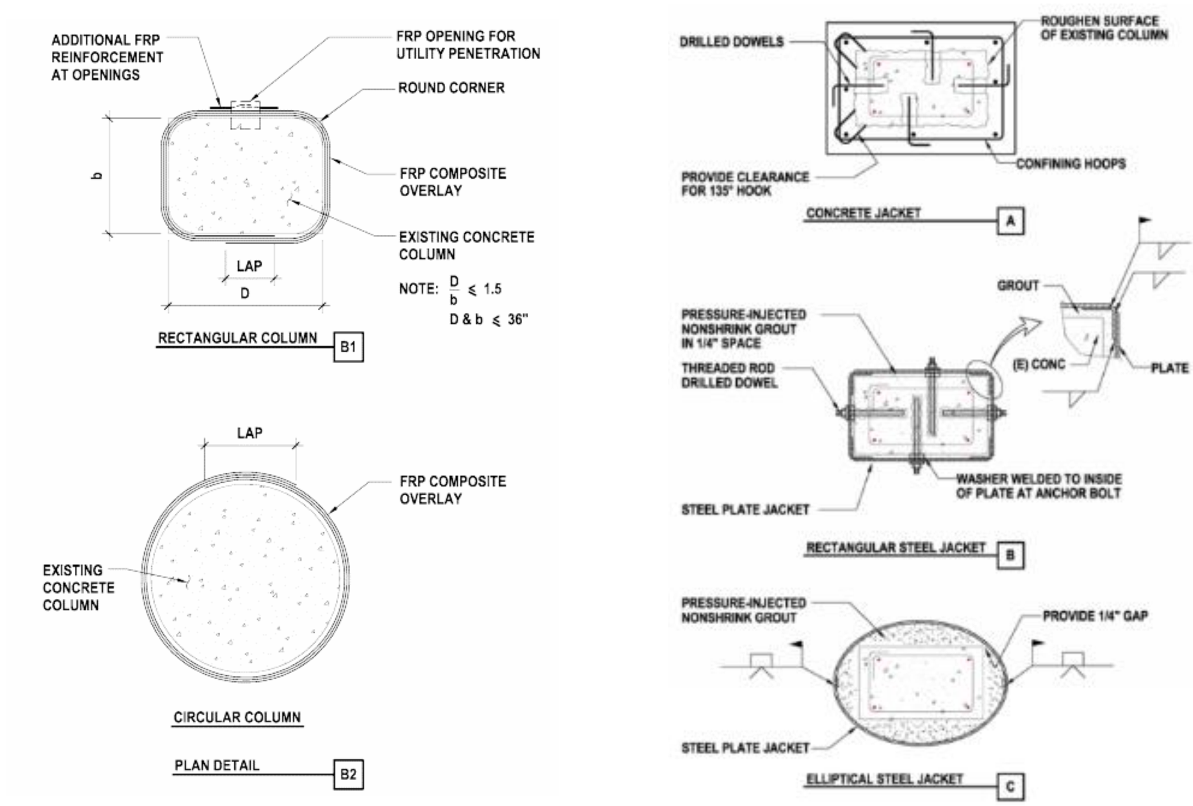
Typical improvements for unbalanced LFRS

Figure 3-36. Typical strengthening for structural deficiencies due to irregularities (FEMA, 2006a)

3.7.1.4 Ductility improvement

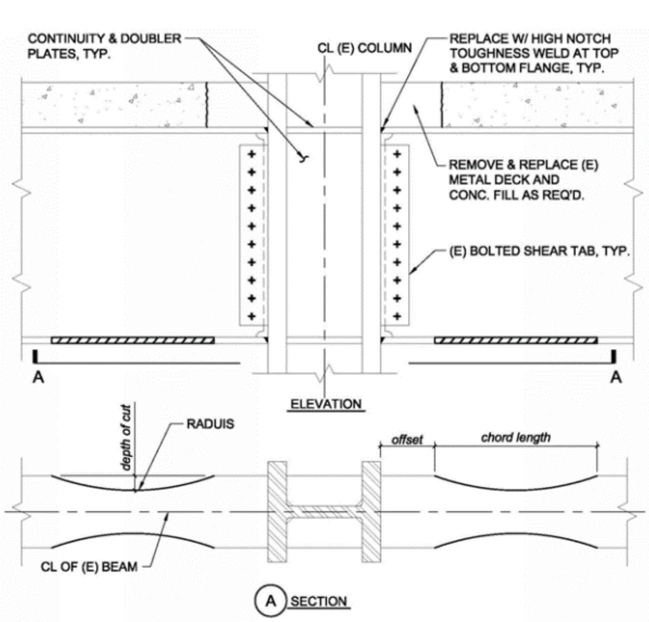
In modern seismic building codes, ensuring a ductile response from buildings during earthquakes has become one of the important design concepts. Ductility is simply described as the capability of large rotation and displacement without any serious damages. Conversely, a brittle response is a sudden failure at small displacement or fragile damage that causes loss in ability to bear gravity loads. Insufficient ductility is, for example, typically caused by an inadequate confinement of structural elements (e.g., column, beam or wall) or an inappropriate yielding location at the column-beam connection.

To ensure the ductile behavior of buildings during earthquakes, some retrofits should be implemented on local elements. For concrete columns or masonry piers, as presented in Figure 3-37, FRP overlay, concrete jacketing or steel jacketing are popular measures to improve the confinement of existing columns, to increase the capacities for shear and axial loads and to enhance ductility. For steel beam-column connections, ensuring a plastic hinge at a ductile location (i.e., not at welding/bolting location) is the important approach to withstanding the inelastic demand of rotation and displacement for ductile response. For this approach, reducing the beam section or adding haunch components, as shown in Figure 3-38, are effective measures for enhancing the plastic moment capacity of beams and the ductile beam response.

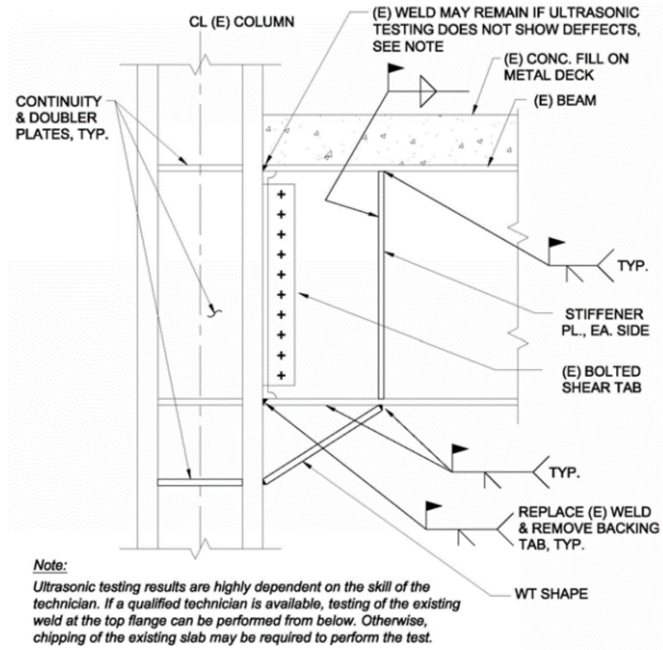


RC column retrofit by FRP overlay RC column retrofit by concrete/steel jacketing

Figure 3-37. Ductility improvement measures for RC column (FEMA, 2006b)



Reduced beam section of existing beam



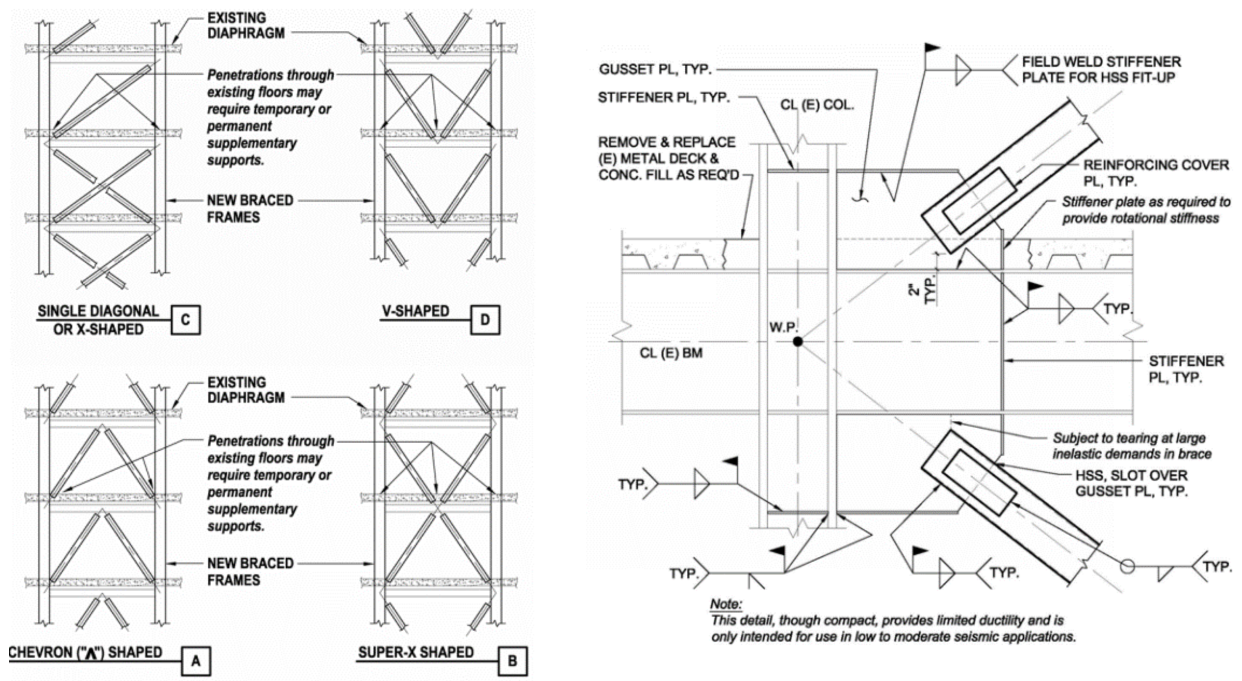
Welded haunch on existing beam

Figure 3-38. Retrofit measures for steel beam ductility (FEMA, 2006b)

3.7.1.5 Stiffness and capacity improvement

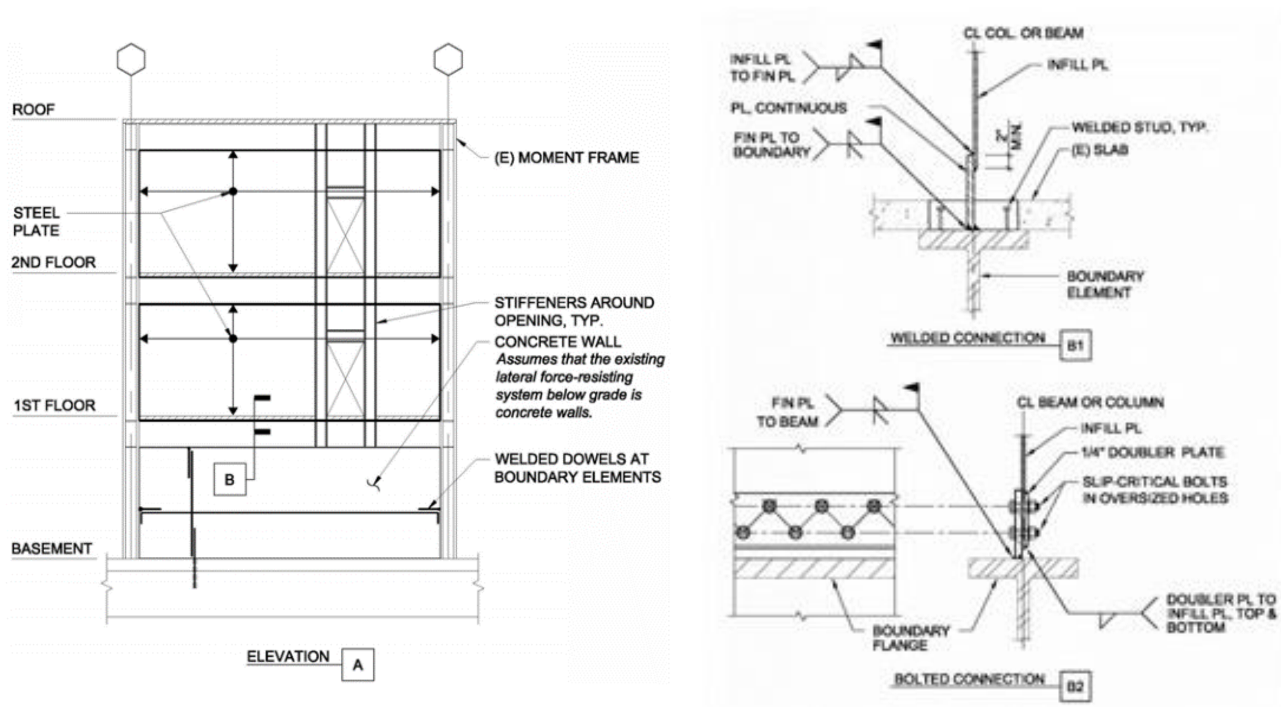
Generally, framed buildings, such as steel or wood structures, are more flexible than walled buildings, such as RC or masonry structures, in terms of lateral stiffness. For those soft buildings, an excessive drift or additional stress due to P-δ effect likely causes structural connection damage, residual displacement and stress on columns, functional damage on nonstructural components, like partition or cladding, and pounding impacts on adjacent buildings.

The installation of new lateral force-resisting elements is a common way to stiffen and strengthen soft buildings. Retrofits with steel brace or steel plate shear wall are shown in Figure 3-39 and Figure 3-40, respectively. Both measures can largely enhance the stiffness of existing framed structures and improve the seismic strength of existing buildings. Usually, new foundations for new braces/walls are needed to transfer the load from the new lateral force-resisting elements to the ground. This type of stiffness improvement works appropriately for wind forces as well.



Brace frame configurations — Typical brace connection on existing elements

Figure 3-39. Steel brace frame retrofit (FEMA, 2006b)



Steel plate shear wall elevation — Typical plate connection on existing elements

Figure 3-40. Steel plate shear wall retrofit (FEMA, 2006b)

3.7.1.6 Energy dissipation system

In this section, two relatively new technologies to improve the seismic performance of buildings are introduced. The first new technology is a seismic isolation system, and the second is a damper system for seismic energy dissipation. Both technologies aim to reduce the seismic demand (i.e., earthquake force) on the buildings by installing devices that would allow for the existing seismic strength of buildings to withstand the reduced demand.

A primary concept of a seismic isolation system is to lengthen the building's natural period of vibration and to pull it away from the major frequencies of earthquake shaking in order to reduce the seismic demand imparted from the ground to the building. As shown in Figure 3-41, the isolator device (with an included damper or a separate damper) has a role in lengthening the building vibration period above the device. The isolator is commonly installed at the bottom of the building to maximize the isolation effect on the entire building. However, these days, the isolator can be installed at the middle height of buildings to deal with potential property-line restrictions or architectural/mechanical reasons. In both cases, additional strengthening is needed for the existing elements around and below the isolator, like beams, columns and foundations. Figure 3-42 presents a seismic retrofit example that demonstrates the base isolation system.

The energy dissipation damper system shown in Figure 3-43 adds an extra damping to the existing building. The increased damping decreases the seismic energy on the building and reduces the local forces on the structural elements. The existing capacity of the building could then endure the earthquake impact, but, similar to the seismic isolation system, some strengthening is likely needed for the beams, columns and foundations around the installed dampers. The installation of fluid-viscous dampers for a hospital retrofit is shown in Figure 3-44.

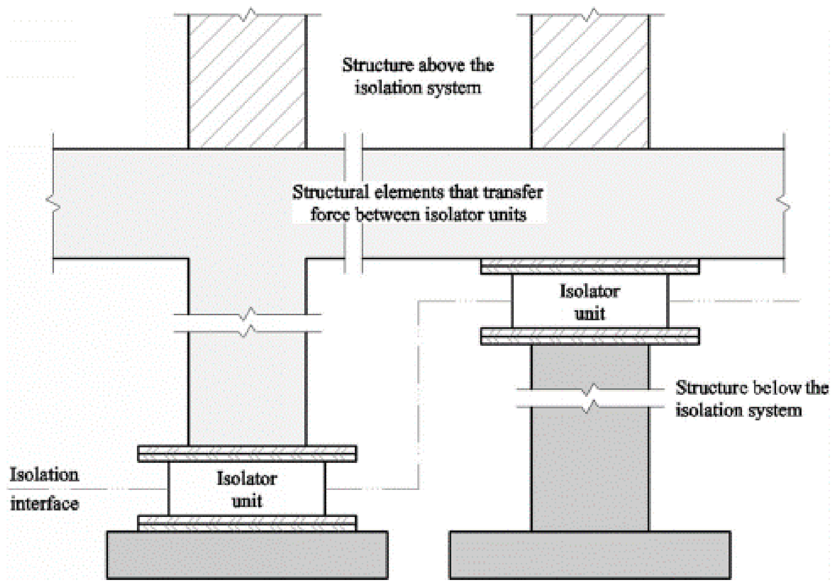


Figure 3-41. Conceptual figure of seismic isolation system (ASCE, 2017b)

Figure 3-42. Base isolation retrofit example of San Francisco City Hall (Naaseh et al., 2006)

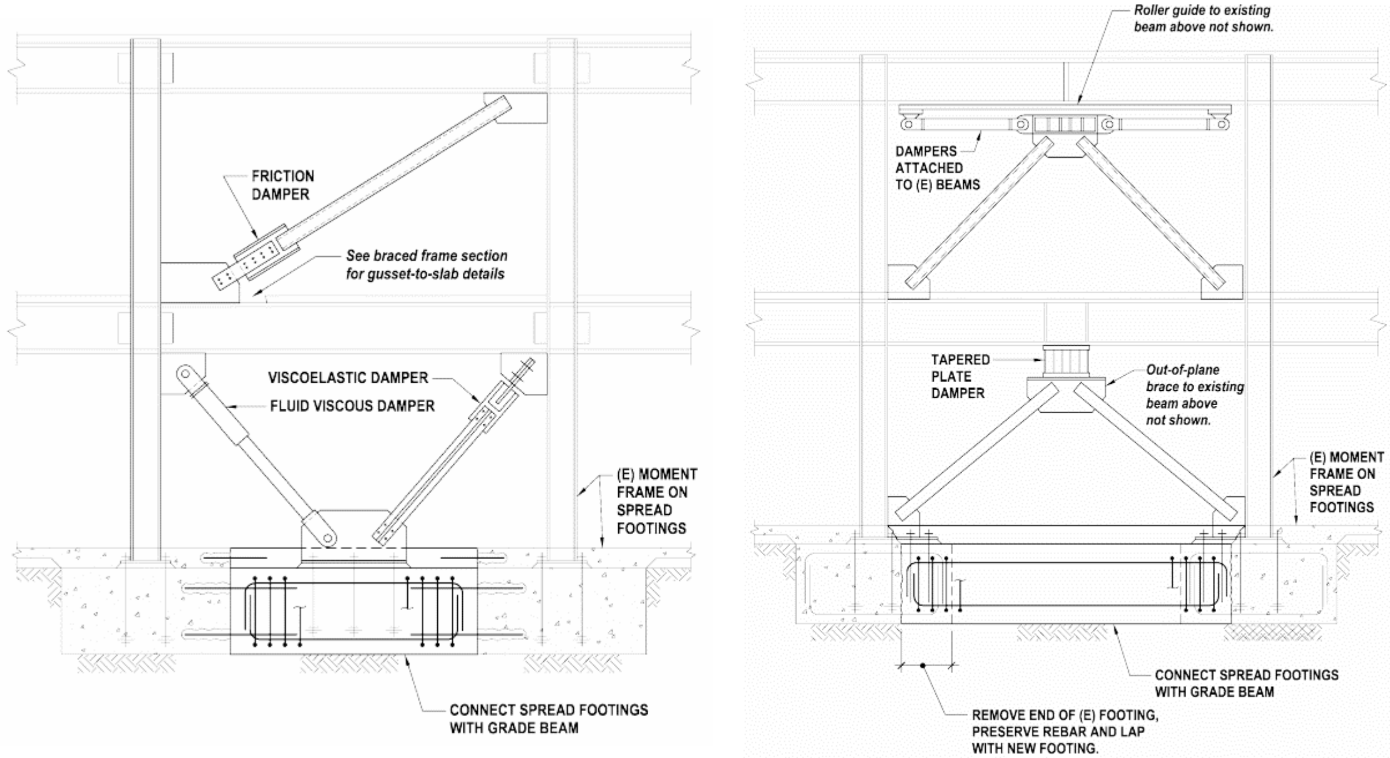


Figure 3-43. Typical configurations of energy dissipation damper retrofit (FEMA, 2006b)

Figure 3-44. Energy dissipation damper retrofit for Naval Hospital Bremerton (FEMA, 2007)

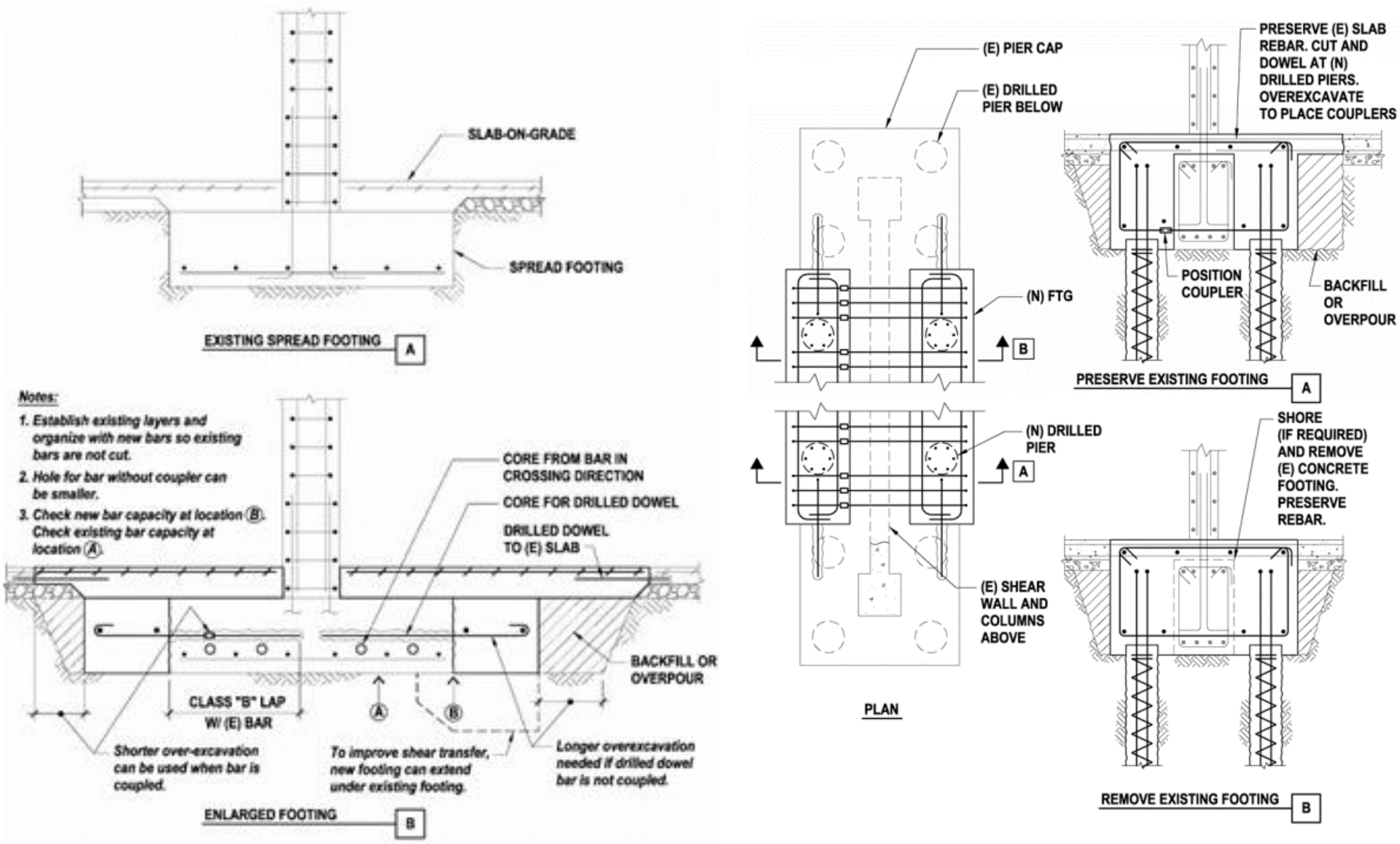
3.7.2 Earthquake: liquefaction

Foundation retrofit and soil improvement are typical approaches to make buildings more robust against earthquake liquefaction. The former is essentially for resisting potential ground failure due to liquefaction and the latter is for improving soil and for reducing liquefaction risk during an earthquake.

3.7.2.1 Foundation retrofit

When the size of shallow-spread foundations or the amount of deep-pile foundations are not sufficient to support the building for a liquefaction event during an earthquake, the consequence could be terrible – tilting, overturning, losing stability or collapse. These damage modes could generate a serious outcome, like demolishment and replacement or extensive repair work with foundation improvement.

For shallow-spread footing, enlarging the existing footing by attaching additional concrete footing is a typical measure to increase capacity; see the left figure of Figure 3-45. New pile installation next to an existing deep-pile foundation is usually implemented to improve the deep-foundation capacity; see the right figure of Figure 3-45. In this case, lateral capacity can be added in addition to compression and tension capacity. In both cases, the retrofit construction work is not simple because, for example, excavation, formwork, reinforcement placement and concrete pouring are usually required around the existing foundation.



Enlarge existing spread footing | Adding piles to existing deep foundation

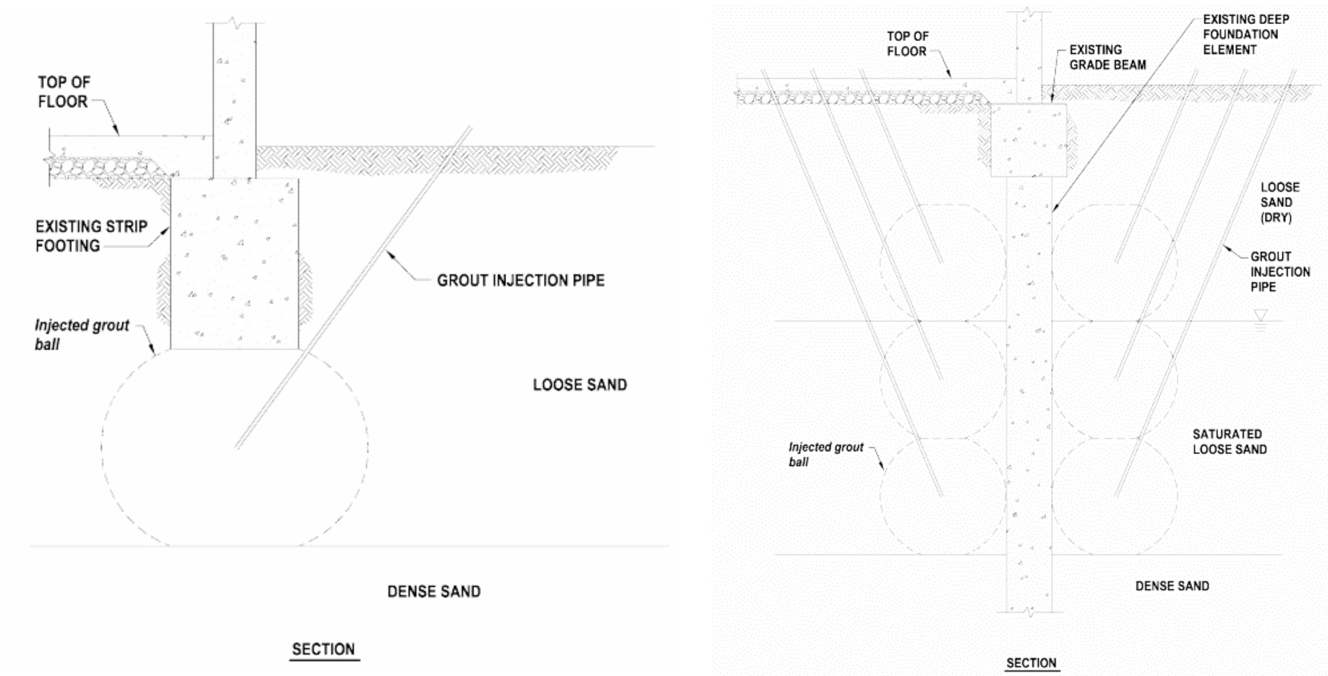
Figure 3-45. Typical foundation strengthening (FEMA, 2006b)

3.7.2.2 Soil improvement

In contrast with the foundation retrofit introduced in Section 3.7.2.1, soil improvement is an alternative to keeping a building stable from large ground failure (i.e., ground settlement and lateral spreading) due to liquefaction.

In Figure 3-46, examples to improve the soil below existing shallow foundations and around existing deep foundations use permeation grouting on liquefiable soil layers. There are generally two types of soil grouting: permeation grouting and compaction grouting. The former injects chemical/cement grout into the soil and aggregate without displacing the materials, and it solidifies the sandy soils and improves resistance to liquefaction. The latter injects very stiff grout into a layer of soil to force the soil particles into a tighter packing arrangement, and it increases the soil density and the liquefaction resistance (FEMA, 2006b).

Lateral spreading usually occurs at the soil above the liquefiable soil layers, and the soil spreads laterally toward an open face when liquefaction happens below. This event might then laterally move the surrounding buildings and destroy the deep-pile foundation of the building. For mitigating the risk of lateral spreading of soil due to liquefaction, the soil grouting measures described above work to prevent liquefaction itself, or the installation of stone/gravel columns by vibrocompaction method to solidify the open face soils. Figure 3-47 shows an example of stone/gravel columns installed around the open face in order to densify the soil layers (FEMA, 2006b).



Under shallow foundation — Around existing deep foundation

Figure 3-46. Permeation grouting on liquefiable soil layers (FEMA, 2006b)

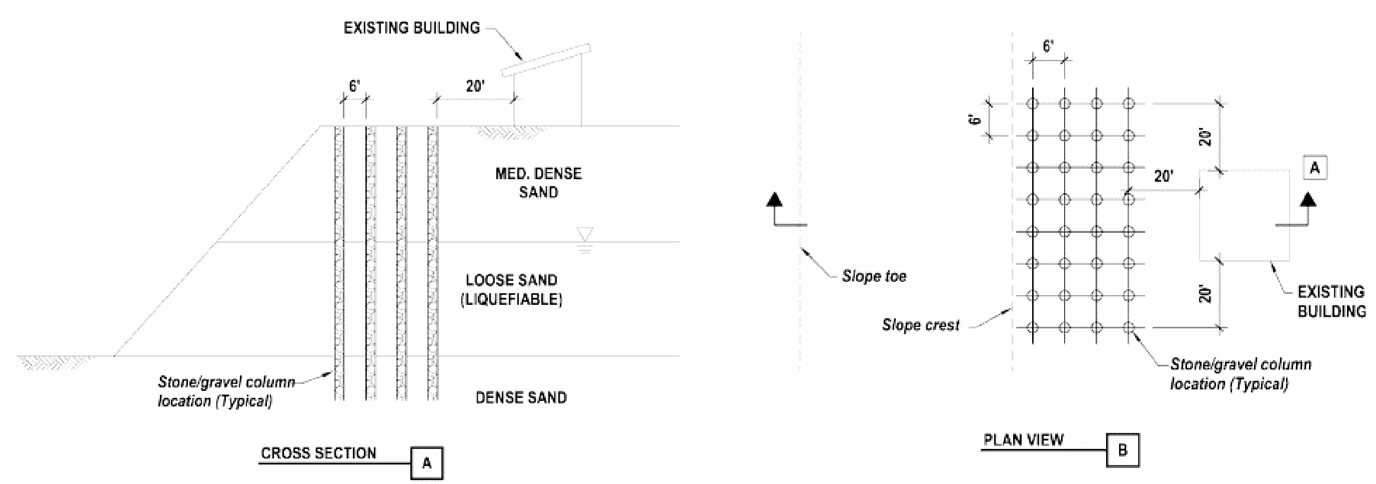


Figure 3-47. Soil strengthening for lateral spreading by stone/gravel column (FEMA, 2006b)

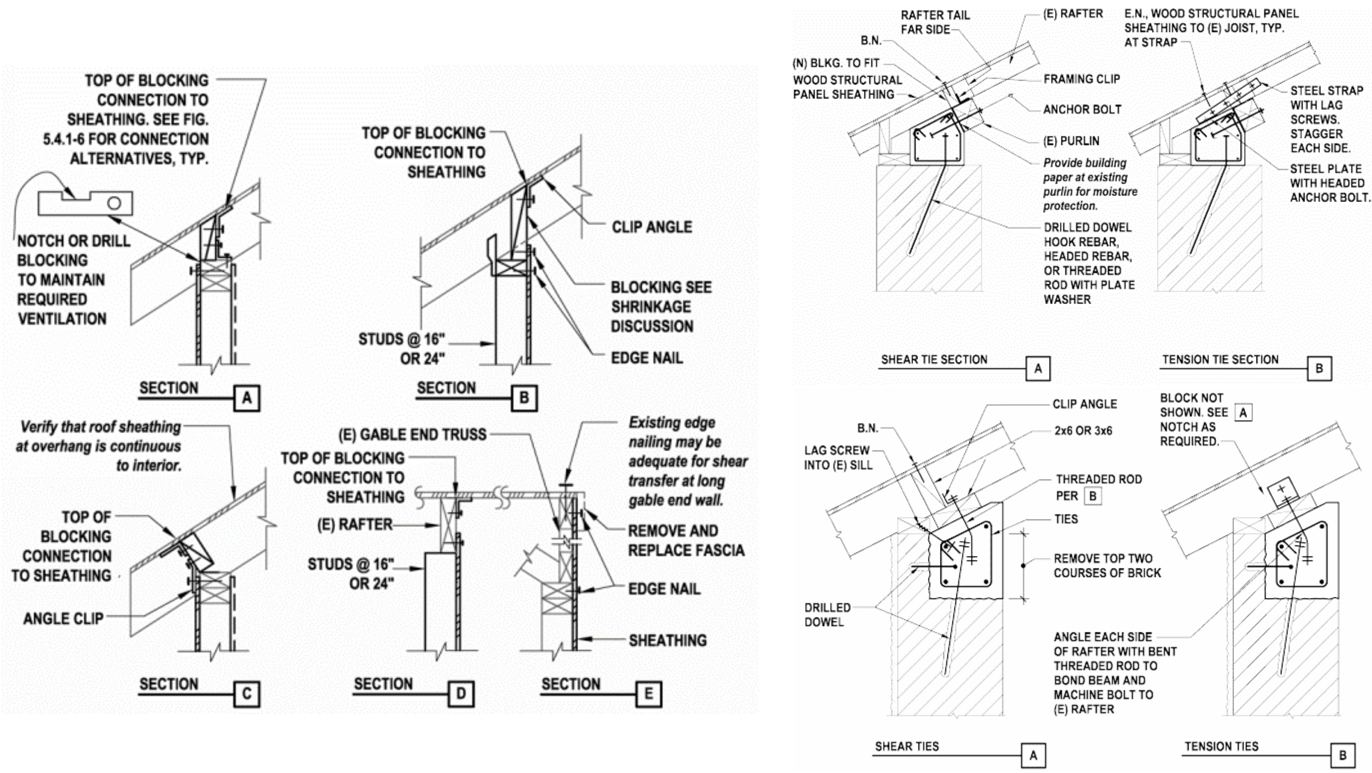
3.7.3 Wind

The connections of roof, foundation and external walls are critical components of buildings for wind hazard. In particular, lightweight structures or their components, like the roof or walls, might be overturned or blown out by strong wind if the connections do not possess sufficient capacity. Typical methodologies to improve those connections are discussed in this section.

3.7.3.1 Roof connection

Strong wind can easily blow the roof away if it is not securely attached to the walls or beams of a building. Since the roof usually has a structural role in horizontally supporting a vertical element such as a wall, losing the roof due to strong wind might cause cascading damage, like wall instability or failure.

Typical details to strengthen the roof connection of wood and masonry structures are shown in Figure 3-48. Adding angle clips and blockings to strengthen the roof connection is a general strengthening measure for wood structures. For masonry structures, new concrete bond beams might be needed to enhance a connection between the masonry wall and roof, and the anchoring, steel strap and nailing are typically provided for securing the roof connection.



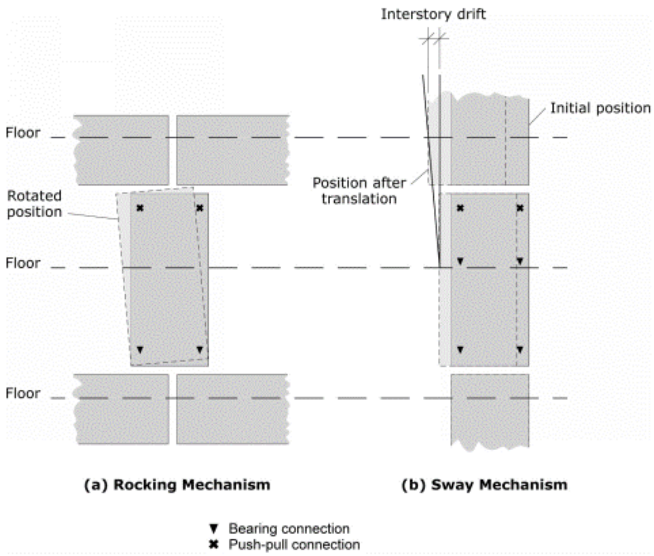
Roof connection of wood structure — Roof connection of masonry structure

Figure 3-48. Typical roof connection improvements (FEMA, 2006b)

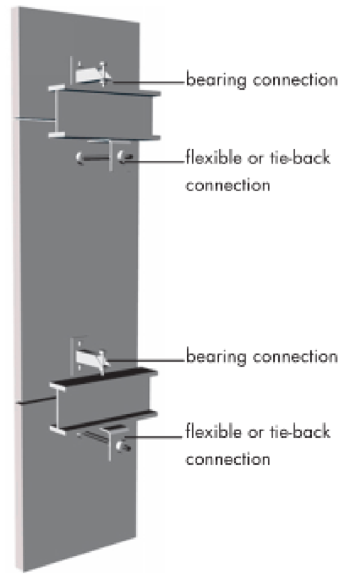
3.7.3.2 External curtain wall and wall

When an external wall is not firmly attached to the building, fall or collapse of external nonstructural walls is a typical failure mode for these components in a strong wind event. If it is not a structural element or bearing wall, it could cause minor damage, structurally, to a building. However, the falling or collapsing element could lead to a serious impact on people or structures around the building.

In modern building codes, particular design mechanisms are required for the connection of external curtain walls, as shown in Figure 3-49. A certain level of capability for rocking and sway should meet the design requirement in accordance with the expected drift and long duration fatigue due to wind. In Figure 3-50, hurricane damage to perimeter block walls and a typical connection detail of block wall, roof and foundation are presented. An adequate connection with anchor and reinforcement could avoid this type of wall failure.



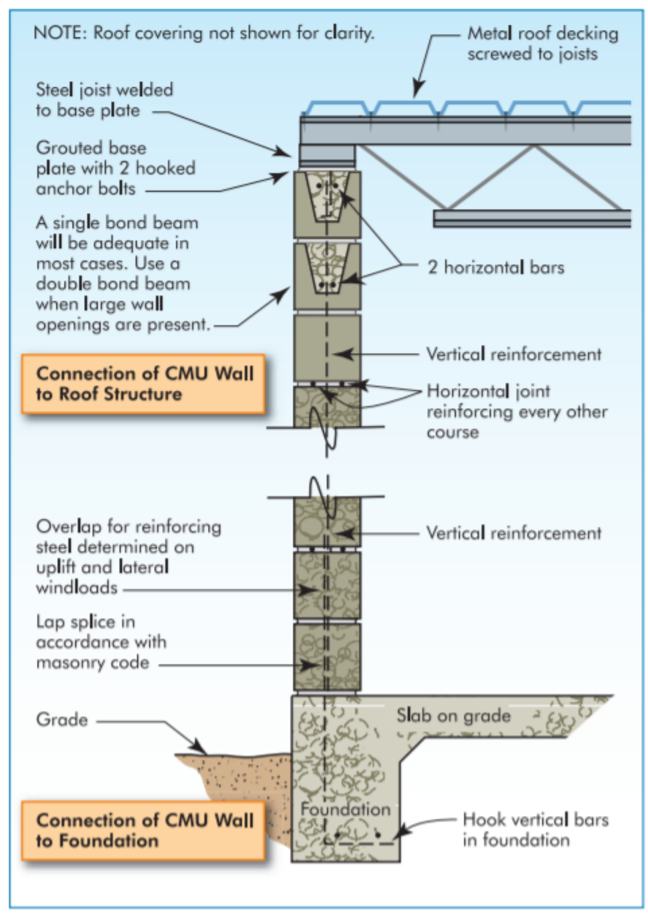
Rocking and sway mechanisms of connection for earthquake (FEMA, 2012)



Push-pull connection of panel (FEMA, 2006a)

Figure 3-49. Typical connection details of external curtain walls

Hurricane damage to external wall



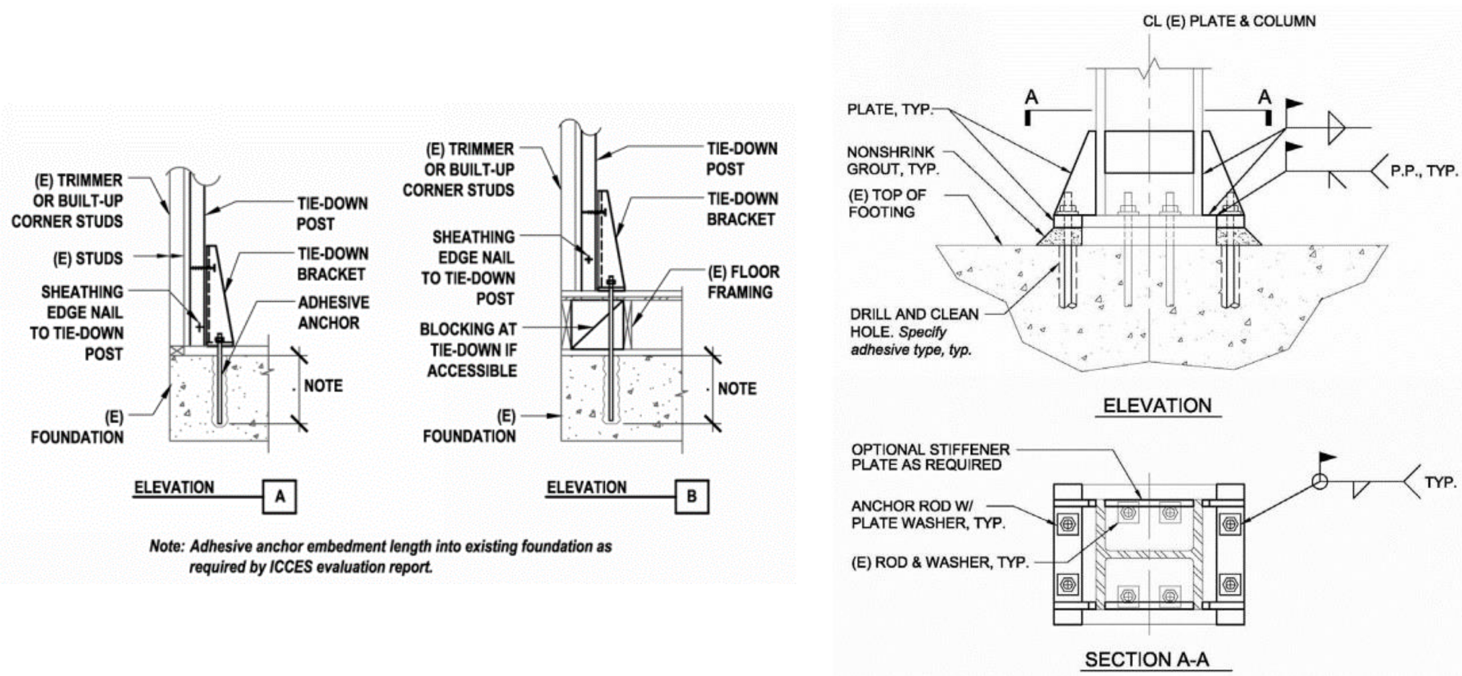
Typical connections of external wall

Figure 3-50. Connections of external wall (FEMA, 2007a)

3.7.3.3 Foundation connection

Uplift and lateral load due to strong wind could cause severe structural damage to a building, like overturning or movement, particularly for lightweight structures like wood and steel-framed buildings. This type of damage likely will induce damages to adjacent buildings as well.

For the rigid foundation connection of wood structures, a hold-down and post-installed anchor with tie-down post should be installed along the existing studs, as shown in the left figure of Figure 3-51. A typical retrofit detail to improve uplift capacity of a steel-column base is presented in the right figure of Figure 3-51. Stiffener plates and base plates are welded to the existing steel elements and post-installed anchors are then installed to the foundation. In both cases, a new foundation could be constructed to resist the large uplift load as needed.



Foundation connection retrofit of wood structure　　Foundation connection retrofit of steel structure

Figure 3-51. Typical connection retrofits of vertical elements to foundation (FEMA, 2006b)

3.7.4 Flood

The general measures to protect buildings from flood hazard aim to prevent flood water inundation and to resist static/dynamic water pressure due to flooding. Water inundation affects the building's functionality or occupancy and water pressure affects the building's structural stability.

3.7.4.1 Building envelope

One of the retrofit measures to keep a building protected from flood inundation is to install floodgates at potential water intrusion routes around the building and flood shields at openings of the building, such as doors and windows; see Figure 3-52, Figure 3-53, and Figure 3-54, respectively. The height of the gate or shield should be designed according to the expected flood depth analyzed by local flood hazard conditions, and the gate and shield need to be designed to resist the water pressure due to inundation depth. These types of building envelope improvements (i.e., watertight system) can also be applied to specific rooms inside of the building, such as machine and electric rooms. Preventing water inundation could help avoid any functional outages of the building that are potentially caused by damage to mechanical and electrical equipment. This outage has to be avoided at essential facilities like emergency centers, government buildings and hospitals.

Figure 3-52. Floodgate installation (FEMA, 2007a)

Figure 3-53. Flood shield for door (FEMA, 2013)

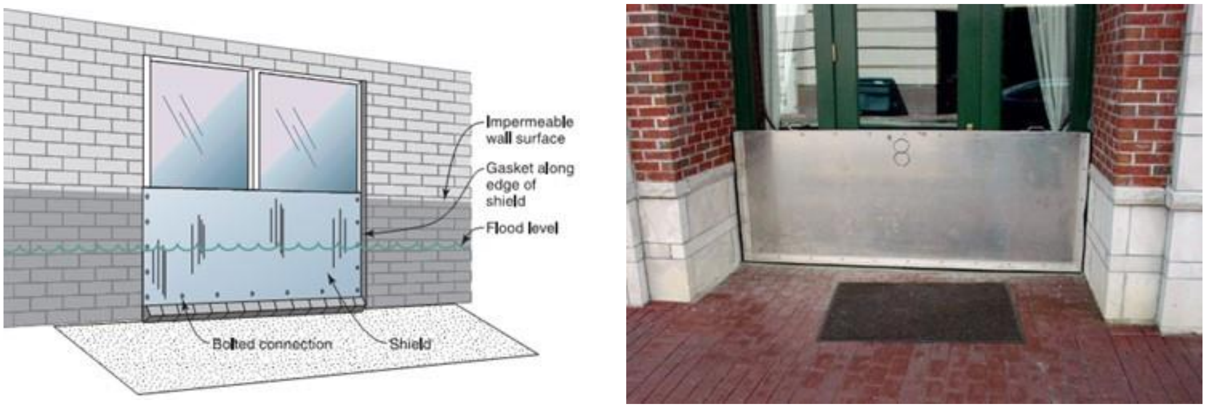


Figure 3-54. Flood shield for window (FEMA, 2013)

3.7.4.2 Elevated construction

Another effective measure to protect a building from flood inundation is to construct an elevated structure, like floodwall around the building. Typical types of floodwalls are shown in Figure 3-55 and represent, essentially, cantilever walls. As a retrofit measure for flood hazard, a floodwall is typically constructed some distance from the building and it should be designed as a freestanding, independent structure separated from the building. This kind of wall is typically constructed of reinforced concrete. For designing a floodwall, the lowest point of the site, topographic information, site-specific flood hazard level (i.e., inundation depth), soil characteristics, potential debris impact and a performance objective are generally identified. As several examples demonstrate in Figure 3-56, a floodwall protects buildings from intrusion of floodwater and provides a barrier against water inundation.

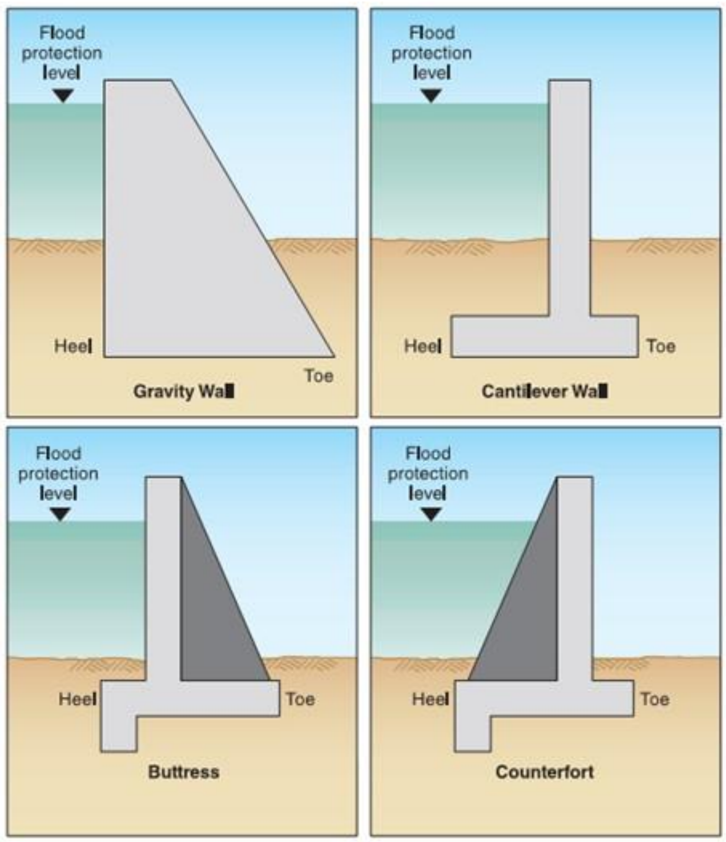


Figure 3-55. Typical floodwalls (FEMA, 2013)

Floodwall at a school

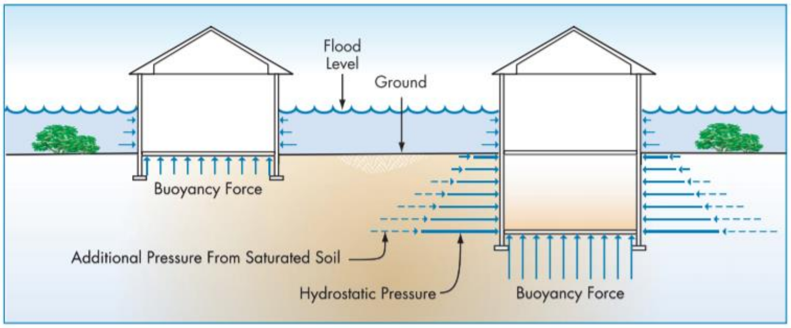
Floodwall at a hospital

Floodwall at a hospital

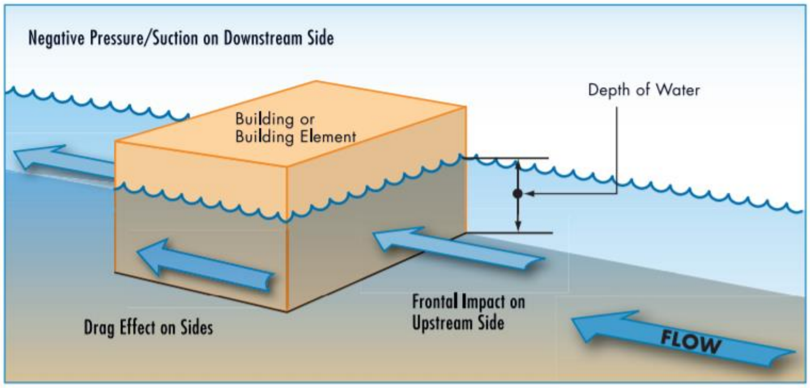
Figure 3-56. Floodwalls protecting buildings (FEMA, 2013)

3.7.4.3 Foundation tie

If a building has watertight or waterproof conditions, the perimeter wall and structural elements of the building have to resist the loads due to flooding water and washed away debris in both horizontal and vertical directions. In Figure 3-57, typical flood loads, such as hydrostatic and hydrodynamic loads, are depicted. In this aspect, the foundation tie for a perimeter wall is a critical component to prevent any severe damage or functional outage of the building. Especially for a lightweight building, a rigid connection to the foundation is very important. A retrofit example of an existing foundation tie for a light wood building is shown in Figure 3-58. Post-installed anchors can be added to the existing connection of wall and foundation.



Hydrostatic loads



Hydrodynamic loads

Figure 3-57. Water loads on building due to flooding (FEMA, 2007a)

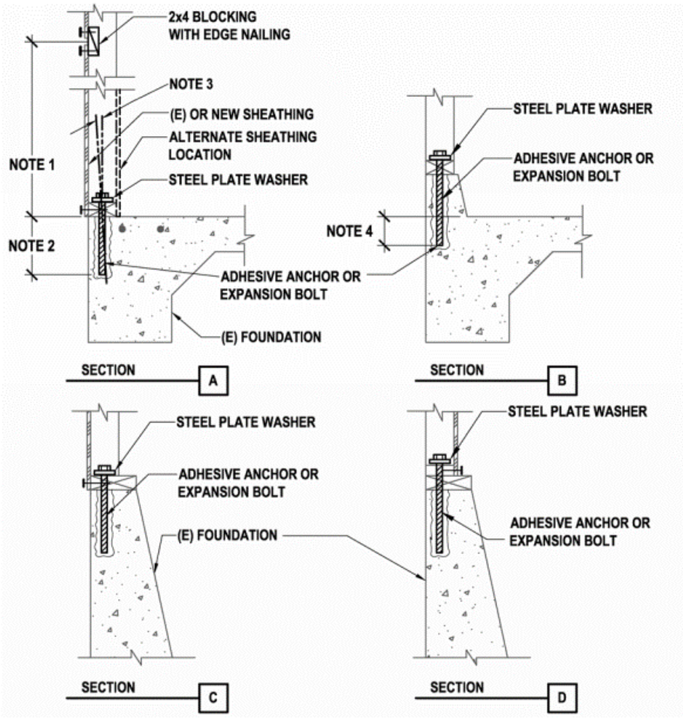


Figure 3-58. Typical strengthening on foundation tie (FEMA, 2006b)

3.8 Cost analysis

3.8.1 Introduction

The service life of critical buildings (i.e., essential facilities) is usually longer than other types of buildings, such as residential, commercial, private office or industrial buildings. For critical buildings, several natural hazard events are anticipated to affect those buildings during their service life. In a natural disaster, critical buildings need to be able to withstand the hazard itself in order to become centers of countermeasures, like an emergency center, evacuation shelter or urgent medical facility. To prevent or reduce the impacts on these important buildings, structural strengthening or repair preparations should be considered in any long-term plan for the maintenance of critical buildings. For example, a periodical condition assessment of structural capacity could be performed that considers building material deterioration recognized by maintenance facilities, and a new hazard map should be updated using the latest research accomplishments. When the necessity of structural strengthening or repair is recognized, improvements have to be implemented. Cost information on improvement measures and building replacement is then indispensable for not only financial planning, but also execution priority.

In this section, cost data and research on building construction and structural retrofits are presented. Information for the Caribbean and the U.S. is then introduced in the reconstruction cost section. To note, retrofit cost data in this section is mainly based on the U.S. construction environment, as similar materials and data are minimal for the Caribbean. However, the ratio of retrofit cost to reconstruction fees is generally expected to be at the same level among many countries, including the Caribbean and the U.S.

3.8.2 Reconstruction costs

The building reconstruction cost (i.e., construction or replacement cost of the building) in the Caribbean is anticipated to be different across the 16 countries studied in this report and from that of the U.S. The data for average construction costs of buildings are summarized in this section.

3.8.2.1 Construction cost in the Caribbean

A financial report for the Caribbean and Latin America provides an indicative construction cost of buildings for some Caribbean countries, as shown in Table 3-23 (BCQS, 2019). The cost is estimated for higher and lower ranges according to property types and includes only the cost of construction material, machinery and labor (i.e., hard cost). The report explains that the cost range is intended to provide data at a 95% confidence interval from the data analysis. The cost ranges (i.e., variation) of the buildings at modest cost are smaller than that of the higher-cost buildings. In this table, the data of hotels/condos and offices, high-rise and special offices could be applied to a cost study of the critical buildings. Critical buildings are usually designed for higher performance level depending on the important usage of the essential facility. Therefore, for example, specialist offices and high-rise story buildings in Table 3-23 could be applied to a cost study of medical facility, government office or emergency center as higher performance and cost are anticipated. Hotels/condos (e.g., three-star level), on the other hand, seem to be adequate for a study of shelter cost because moderate performance and cost are expected for temporal use and evacuation shelter.

3.8.2.2 Construction cost in Saint Vincent and the Grenadines

Specific information for construction, including cost for the critical buildings in Saint Vincent and the Grenadines, is presented in Table 3-24 (Campbell, 2020). The occupancy, footprint area, number of stories, frame type, primary material, number of occupants, etc. are all listed, and costs are shown in Eastern Caribbean dollars (EC$) and U.S. dollars (US$). There are some variations based on building occupancy, and the construction cost per unit area for hospital and government buildings seems to be higher than schools or hospitalities/hotels.

Item	Bahamas		Barbados		Jamaica		St. Lucia		St. Maarten		Trinidad & Tobago	
	Low	High	Low	High	Low	High	Low	High	Low	High	Low	High
Residential high quality	280	460	250	500	180	300	190	350	240	390	175	350
Residential medium quality	160	260	140	230	110	170	125	160	150	230	100	150
Residential modest quality	120	200	120	180	80	130	90	130	120	180	75	120
Hotels/condos five star	330	550	310	480	220	360	285	350	290	460	250	330
Hotels/condos three star	170	290	190	250	120	190	175	235	160	250	170	250
Offices 1-3 story, shell	150	240	160	230	100	160	140	200	140	210	130	175
Offices 1-3 story, fitout	270	450	250	400	180	300	190	300	240	380	170	250
High rise 4-8 story, shell	220	370	190	250	150	240	160	230	200	310	155	220
High rise 4-8 story, fitout	330	550	290	450	220	360	240	340	290	460	220	290
Specialist offices	340	570	300	500	230	370	275	400	290	470	225	375
Storage or warehouse	80	120	75	110	50	80	70	100	80	120	60	90
Retail single story	130	210	135	180	80	140	110	140	120	190	100	150

Table 3-23. Construction cost of buildings in the Caribbean in US$ per Sq. Ft. (BCQS, 2019)

Type	Footprint (m^2)	Stories	Framing type	Primary material	Occupants	Year built	Design code	Construction Cost			
								EC$	EC$/ m^2	US$ /m^2	US$ /ft^2
School	2,441	one	Concrete frame	Concrete and Mason Block	484	2020	ACI-318-14	2,997,827	1,228	454	42
School	331	one	walls	Concrete	49	2017	ACI-318-14	47,523	144	53	5
Hospital	607	two	Concrete frame	Concrete and Masonry Block	5,756	2019	ACI-318-14	2,200,000	1,813	671	62
Hospital	718	one	Concrete frame	Concrete	7,061	2013	ACI-318-14	924,000	1,287	476	44
Government	505	one	Concrete frame	Concrete	5,300	2017	ACI-318-14	1,400,000	2,770	1,025	95
Hospital	697	one	Concrete frame	Concrete	5,300	2015	ACI-318-14	1,120,000	1,606	594	55
Hospitality/ hotel	1,621	two	Concrete frame	Concrete, wood	251	2019	ACI-318-14	675,000	208	77	7

Table 3-24. Construction cost of essential facilities in Saint Vincent and the Grenadines (Campbell, 2020)

3.8.2.3 Construction cost in Belize

A report for a workshop under the U.S. Agency for International Development/Organization of American States (USAID/OAS) Post-Georges Disaster Mitigation Project documents the average construction cost of residential buildings in Belize, as shown in Table 3-25. According to the household income level (i.e., low, middle or upper), the average unit area and construction cost of residential building was summarized with some variations. The unit construction cost (US$/Sq. Ft.) of upper income residences was approximately two to three times more than that of low income residential building in Belize. For the average area of a residential unit, there is about a seven times difference among the two classes.

Income	Area (Sq. Ft.)	Construction rate (US$/Sq. Ft.)	Cost range (US$)
Low	360 – 640	30 – 50	10,800 – 32,000
Middle	640 – 2,200	50 – 65	32,000 – 143,000
Upper	2,200 – 5,000	65 – >100	143,000 – >500,000

Table 3-25. Construction cost of residential buildings in Belize (Henderson, 2001)

3.8.2.4 Construction cost in the U.S.

The typical construction costs for buildings in the U.S. are introduced here as a reference for this study. In the U.S., this type of cost varies between different regions or cities. A construction market analysis report for the U.S. investigated the average construction costs according to major cities and building occupancy; see Table 3-26 (Cumming, 2020). The costs shown in the table are the total cost, combining hard cost (e.g., material fees and labor cost) and soft cost (e.g., plan/design fee and inspection fee), and they are at the same level as construction cost analyzed by other researchers (Raetz et al., 2020). Similar to other cost estimations shown in this section, the cost statistics also indicate some variations (i.e., lower and higher limits). Five major cities on the West Coast of the U.S. were chosen to reference because earthquakes are much more frequent here than other U.S. cities, and wind and flood disasters happen here as well. The construction costs of healthcare, higher education and public/community facilities seem to be relatively higher than that of other occupancy types based on the cost in Table 3-26.

Item	San Francisco		Los Angeles		San Diego		Seattle		Portland	
	Low	High	Low	High	Low	High	Low	High	Low	High
Residential										
Single Family Medium Quality	322	386	65	241	139	246	258	310	245	294
Apartment/Condo. Mid-rise	449	584	294	529	273	503	361	469	343	445
Commercial/Office										
Single Story	364	437	327	395	314	381	292	350	277	333
Mid-Rise	726	870	371	567	359	466	501	629	476	597
High-Rise	833	1,001	458	688	430	678	561	696	533	661
Hospitality/Lodging										
Three-Star Hotel	555	731	425	661	348	482	376	452	358	429
Five-Star Hotel	761	1,044	679	945	456	712	591	838	561	796
Healthcare										
Acute Care Facility	1,075	1,433	973	1,296	929	1,238	566	782	538	743
Medical Office Building	612	734	498	598	476	571	398	493	378	469
Specialty Clinic	765	918	623	748	595	714	553	663	525	630
Primary/Secondary Education										
Elementary School	329	392	341	410	326	391	242	291	230	276
Middle School	362	431	374	448	357	428	265	318	252	302
High School	395	475	411	493	392	471	291	350	277	332
Higher Education										
Academic/Classroom	618	742	645	774	616	739	515	617	489	587
Laboratory	640	823	801	962	637	816	711	853	675	811

Administration	628	742	666	799	626	735	532	638	505	606
Dormitory	344	411	401	426	343	407	276	340	262	323
Public/Community Facilities										
Gov't Administrative Buildings	474	767	627	759	463	599	557	673	529	640
Museum/Performing Arts	727	1,272	921	1,151	707	1,099	817	1,021	776	970
Recreation/Gymnasium	438	546	412	494	386	490	365	438	347	416
Police Stations	590	822	599	718	572	686	531	637	504	605

Table 3-26. Construction cost of buildings in the U.S. in US$ per Sq. Ft. (Cumming, 2020)

3.8.3 Strengthening costs

As cost information for strengthening of existing buildings in the Caribbean was scarce, the available information for retrofit cost in the U.S. (mainly for earthquakes) was then included in this section, as it is anticipated that the ratio of strengthening-to-reconstruction cost of buildings is similar between the Caribbean and the U.S.

3.8.3.1 Methodology for estimating seismic retrofit cost

A methodology to estimate typical costs for seismic retrofit of existing buildings was developed and published in FEMA 156 and 157 (FEMA, 1994)(FEMA, 1995). For the methodology of FEMA 156 and 157, a database of seismic retrofit costs for various types of buildings in the U.S. was provided and a regression analysis was conducted to form the formula to evaluate seismic retrofit cost.

A similar research project was performed later based on the methodology of FEMA 156 and 157. The project added additional and more recent data on retrofit cost of buildings to refine the estimation formula of FEMA 156 and 157 (NIST, 2017). For a number of building types, listed in Table 3-27, regression analyses based on the database and several trials with different parameter combinations were carried out. The formula most aligned with the actual cost database was then developed, as shown in Eq. 3-4. As seen in the formula, several factors (seismicity, performance objective, area, age, stories, occupancy, and historic variables) are considered. The estimated coefficients for this retrofit cost formula are presented in Table 3-28. The author of the report states that this formula was developed for estimating the seismic retrofit cost of existing federal buildings, but that the development approach can be generalized to any kind of building inventory.

Eq. 3-4 $$ln(C)_{s,p,b} = \alpha + \eta_s + \delta_p + \zeta_{sp} + \gamma_b + \beta_1 ln(Area) + \beta_2 ln(Age) + \beta_3 ln(Stories) + \beta_4(Occupancy\ condition) + \beta_5(Historic) + \varepsilon$$

- C: Structural cost per square foot of retrofitting a particular building,
- s: denotes Seismicity of the building,
- p: denotes Performance objective of the retrofit,
- b: denotes Building group,
- α: Performance objective of the retrofit,
- η_s: Fixed effect of seismicity,
- δ_p: Fixed effect of performance,
- ζ_{sp}: Combined fixed effect (interaction) of seismicity and performance,
- γ_b: Fixed effect for building group,
- $\beta_{1\ldots5}$: Model coefficients (regression variables),
- ε: Unobserved error term.

Building group	**Model**	**Building types**
1	URM	Unreinforced masonry
2	W1	Wood light frame
	W2	Wood (commercial or industrial)
3	PC1	Precast concrete tilt up walls

	RM1	Reinforced masonry with metal or wood diaphragm
4	C1	Concrete moment frame
	C3	Concrete frame with infill walls
5	S1	Steel moment frame
6	S2	Steel braced frame
	S3	Steel light frame
7	S5	Steel frame with infill walls
8	C2	Concrete shear wall
	PC2	Precast concrete frame with concrete walls
	RM2	Reinforced masonry with precast concrete diaphragm
	S4	Steel frame with concrete walls

Table 3-27. Building group and types for the prediction model (NIST, 2017)

Parameter	**Dependent variable: Cost per sf**
	Main model
Area	-0.164 (0.038)
Age	-6.080 (4.680)
Stories	0.306 (0.068)
Occupancy[6]: TR	-0.308 (0.096)
Occupancy: IP	-0.947 (0.119)
Seismicity[7]: M	-0.064 (0.404)
Seismicity: H	-0.038 (0.337)
Seismicity: VH	-0.132 (0.322)
Performance[8]: DC	0.040 (0.371)
Performance: IO	0.635 (0.479)
BG: 2	-0.129 (0.174)
BG: 3	-0.139 (0.191)
BG: 4	0.326 (0.139)
BG: 5	0.330 (0.208)
BG: 6	-0.668 (0.254)
BG: 7	0.505 (0.177)
BG: 8	0.145 (0.141)
Historic	0.975 (0.153)
M x DC	-0.095 (0.464)
H x DC	-0.127 (0.484)
VH x DC	0.325 (0.395)
M x IO	-0.512 (0.560)
H x IO	-0.336 (0.581)
VH x IO	0.271 (0.500)

[6] Building occupancy condition: In-place (IP) and Temporarily removed (TR) during retrofit work
[7] Seismicity of building location: Very high (VH), High (H) and Medium (M)
[8] Performance objective: Immediate occupancy (IO) and Damage control (DC)

Constant	50.400 (35.400)
Observations	812
R^2	0.321
Adjusted R^2	0.300

Table 3-28. Estimates of coefficients and variations from training main model (NIST, 2017)

3.8.3.2 Cost-benefit analysis of seismic retrofit for URM buildings

A cost-benefit analysis for the seismic retrofit of URM buildings in Portland was performed to support the city's decision-making on a seismic retrofit ordinance. Existing URM buildings are usually outdated with insufficient structural seismic capacity and are therefore very vulnerable to earthquake hazards. In Portland, 1,661 URM buildings existed at that time, as mapped in Figure 3-59.

In the analysis, the URM buildings were classified into six classes according to occupancy and performance level; see Table 3-29. Class 1 buildings are the most critical buildings in an earthquake event, and Class 5 buildings are not essential facilities in post-earthquake circumstances and their performance objective is thus at the lowest level. The retrofit costs applied in the study considered the construction costs, relocation costs and usual soft costs (e.g., design fee, etc.) for each building class, as listed in Table 3-30. For the benefit evaluation, the reduced damage amount of buildings and contents, the decreased number of human losses and the diminished displacement cost (i.e., cost for evacuation and moving due to building damage) because of seismic retrofitting were converted to monetary values and considered in the analysis.

In the study, the cost-benefit ratio was calculated as the net present value of future benefits divided by the seismic retrofit cost. The analysis was performed for a 50-year building lifetime and used the average annual damages and losses of both the existing and retrofitted buildings. The ratios of benefit-to-cost for seismic retrofits for each building class were then analyzed, as summarized in Table 3-30. All calculated ratios are larger than 1.0, which means that seismic retrofit is beneficial for all building classes. There were no clear explanations for the "N/A" specification for class 1 buildings in the original document, but it can be assumed, based on engineering judgement, that seismic retrofit was undoubtedly profitable for the class 1 buildings because they are the most critical buildings during an earthquake disaster.

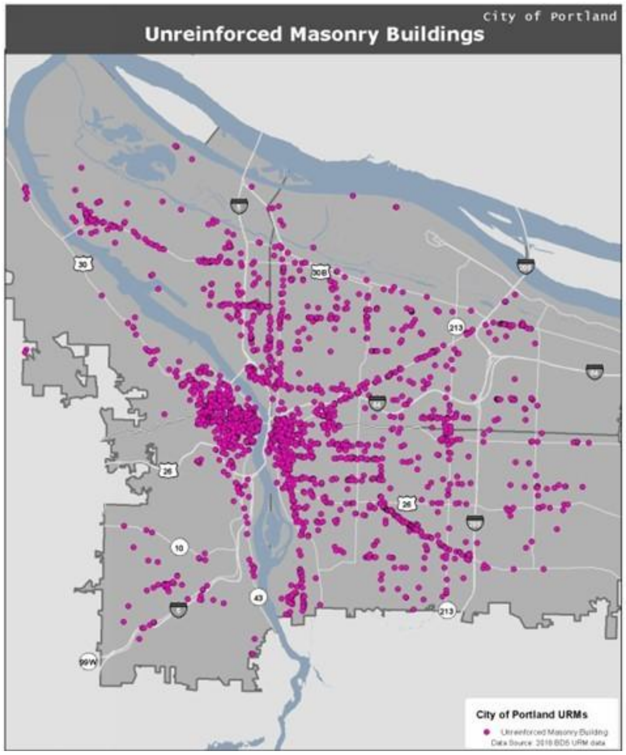


Figure 3-59. Geographic distribution of URM buildings (purple dot) in Portland (Goettel, 2016)

URM class	Count	Percent	Simplified definition	Seismic performance level (simplified)
Class 1	10	0.60%	Critical facilities	Comparable to a new critical facility
Class 2	92	5.54%	Schools and public assembly buildings	Damage control for 20% in 50-year ground motion and life safety for 5% in 50-year ground motion
Class 3	220	13.25%	Buildings with ≥4 stories, or ≥300 occupants or residential buildings with ≥100 units	Life safety for 20% in 50-year ground motion and collapse prevention for 5% in 50-year ground motion
Class 4-A	1,126	67.79%	"Ordinary buildings" not in the other classes	Same as Class 3, unless exception applies
Class 4-B				Very similar to Class 5, if exception applies
Class 5	213	12.82%	1- and 2-story buildings with 0-10 occupants	Bolts Plus, retrofit of only specified building elements

Table 3-29. URM building classes and performance levels in Portland (Goettel, 2016)

Portland URM class	Estimated retrofit cost per square foot (in USD)	Typical building benefit-cost ratio
Class 1	US$111.45	N/A
Class 2	US$82.62	1.474
Class 3	US$68.77	1.661
Class 4-A	US$68.77	1.661
Class 4-B	US$51.00	1.967
Class 5	US$20.00	1.940

Table 3-30. Building group and types and benefit-cost ratios for the prediction model (Goettel, 2016)

3.8.3.3 Retrofit cost consideration in FEMA documents

In documents published by FEMA (FEMA, 2006b)(FEMA, 2007a)(FEMA, 2012)(FEMA, 2013d), there are considerations of the retrofit cost, disruption matter and construction approach based on U.S. practices. The summarized commentaries for typical strengthening measures introduced in this study are presented in Table 3-31.

Retrofit	Option	Description
Wall out-of-plane strengthening	Diagonal or vertical bracing (FEMA 2006b)	Diagonal bracing is usually less expensive but is considered less reliable than vertical bracing. Exposed braces are typically less expensive than more architecturally-sensitive alternatives like recessed vertical braces or reinforced cores. Installation of bracing is fairly disruptive, since it must occur around the entire perimeter; and it involves drilled dowels and accessing and connecting to horizontal diaphragms.
In-plane strengthening, Structural irregularity, Stiffness and Capacity improvement	RC shear wall (FEMA 2006b)	In general, shotcrete walls are less expensive than cast-in-place concrete because at least one side of the wall forming is eliminated. Construction of new shear walls in an existing building can be very disruptive to any building occupants. Noise, vibration, and dust associated with many operations, especially cutting holes through and drilling dowels into concrete, can be transmitted throughout a concrete structure. Placing cast-in-place concrete or shotcrete is a wet process and very messy. Also, excavation and drilling operations and the use of mechanized and/or truck mounted equipment associated with installation of new foundations can be very disruptive
	Steel brace (FEMA 2006b)	Costs will be less when existing moment frames are converted into braced frames to take advantage of the existing strength and stiffness of the frame members, connections, and foundations. Designs that are simple and details that are not overly complicated will also minimize costs. Costs can also be reduced if disruption is minimal during construction. Installing braces at the perimeter frames reduces logistical issues associated with working in confined spaces and temporary removal of the nonstructural elements. Noise associated with this type of work is loud and disturbing to the tenants if the building is occupied while the work is being performed.
	Steel plate wall (FEMA 2006b)	Unstiffened walls are cheaper and less labor intensive than stiffened walls. New foundations are almost always required for new walls and could be extremely costly if deep foundations, such as drilled piers, are added. Installing new walls is disruptive to the occupants because of the noise and vibrations associated with construction. Even if tenants are relocated to parts of the building where the work is not being performed, vibrations associated with cutting, chipping, and drilling of concrete as well as the installation of steel panels can transmit through the structure. The disruption can be reduced somewhat if the walls are installed at the perimeter
	FRP sheet overlay (FEMA 2006b)	In addition to the costs of material and installation of FRP composite, concrete surface preparation and final appearance requirements must also be considered. If additional surface finishing is required, such as paint or a cementitious appearance, this can add significant cost, and should be coordinated with the architect and owner. Although the basic materials are generic, the fabric that will be supplied is proprietary. Strand orientation and density, epoxy overlays, preparation requirements, and installation procedures may be different between suppliers. Some suppliers may not have experienced applicators in the area of the project. These variations must be considered to achieve an adequate specification, particularly if competition between suppliers is desirable.
	RC wall overlay (FEMA 2006b)	Adding new concrete, particularly with shotcrete, can be quite disruptive. Where access is sufficient, shotcrete is typically chosen as it is less expensive than cast-in-place work which requires front-side formwork. There are no known proprietary concerns with shotcrete or cast-in-place overlays on existing masonry walls.
Ductility improvement	Concrete/Steel jacketing on RC column (FEMA 2006b)	Adding a concrete or steel jacket is a more traditional method of enhancing a deficient concrete column. Because of the difficulty of placing an overlay on all sides of an existing column, concrete overlays are typically done with cast-in-place concrete, rather than shotcrete. The need for formwork is a significant disadvantage for concrete overlays, compared to FRP and steel overlays. Placing the concrete and vibrating are also challenging due to access limitations at the top of the column where it runs into beams or slabs. Pour ports or holes in the diaphragm are needed. Steel overlays are typically 3/16" or 1/4" thick and become quite heavy. Access and lifting issues in existing buildings can force the overlay to be broken down into

Retrofit	Option	Description
		pieces, increasing the amount of field welding necessary to join the pieces back together. Unlike FRP overlays, no proprietary issues have been identified with using concrete of steel jackets.
	FRP sheet overlay on RC column (FEMA 2006b)	To appropriately evaluate the cost of a retrofit scheme using FRP overlay in comparison to traditional retrofit concepts (such as concrete or steel jacketing), one needs to consider the cost of the raw material, the level of specialization required by the contractor to install the system, the cost of labor and equipment, the cost of quality control and quality assurance, the temporary impact of disruption during construction, and the permanent impact to the building functions. Although FRP overlays are relatively expensive compared to steel and concrete, they can offer advantages when only limited access is available or minimal disruption of existing conditions is desired.
	Steel beam connection (FEMA 2006b)	Connection modifications are locally very disruptive. Modification of a connection typically requires access from two floors to perform the work on each flange. Noise associated with this type of work will spread and disrupt tenants on other floors unless the work is done during off-hours. The RBS (Reduced Beam Section) is probably cheaper than the other modifications since it requires the least amount of material and labor. With older buildings, there may be asbestos present in the fireproofing around the steel members, which could require costly abatements to expose the connections.
Energy dissipation system	Seismic isolation (FEMA 2006b)	Detailing for rubber and sliding bearings (i.e., isolator) is quite different. If multiple vendors are necessary, vendors of different types of systems are usually considered. Often, they are procured in an early package, due to the long lead time. This also permits the design engineer to move into final design knowing which type of system will be used. In addition to the bearings and dampers, a complete isolation system will require a number of other special elements, including a moat around the building to accommodate the displacements. The moat has to go down past the plane of isolation. Elevators are typically hung from the superstructure, as they cannot cross the isolation plane without special detailing. Utilities entering the building need to be able to accommodate the isolation displacements; this often triggers special vaults outside the building or areas under the building for joint details. A foundation is needed below the isolators to take the forces they impart, and a structural system is needed above the isolators as well to deliver forces to the isolators and resist the moments that are induced. All of these elements add to the cost of isolating the building.
	Energy dissipation damper (FEMA 2006b)	Most dampers available on the market are proprietary. Material properties, testing histories, limitations and detailing considerations are obtained from the manufacturer. Like seismic isolation components, the particular category of damper such as a fluid viscous damper or a friction damper is usually selected early in the design because the analysis and detailing can be significantly different between categories. There is also a patent regarding certain techniques for connecting bracing and dampers to beams when sliding is employed. The damper must be connected to the existing structure and potentially the foundation. When dampers are added to the structure, the loads they impart locally must be considered in the design.
Foundation retrofit	Enlarge shallow foundation (FEMA 2006b)	Enlarging or replacing an existing footing is a localized but disruptive process, involving excavation, dust, mud, drilling/jackhammering noise and concrete placement. Protection of existing finishes in the vicinity and in the working path is necessary. If the existing footing is replaced, shoring will be needed. It is critical that the base of footing be properly compacted, and the new concrete be tightly placed beneath the existing column to minimize or eliminate any settlement when the shores are removed.
	Add pile deep foundation (FEMA 2006b)	Adding new drilled piers in an existing building is very disruptive, involving excavation, dust, mud, drilling/jackhammering noise and concrete placement. Protection of existing finishes in the vicinity and in the working path is necessary. Drilling limitations for drilled piers may need to be applied according to site environment. Drilled piers typically have spacing limits of three times the pier diameter to avoid group effect reductions. This can limit the number of piers that can be installed.

Retrofit	Option	Description
Soil improvement	Grouting around existing foundation (FEMA 2006b)	Grouting can be quite disruptive and costly especially if injection holes have to be drilled through existing concrete footings and slabs. If current operations in the building are to continue, the usual approach is to do the grouting at night and clean up the work area before the start of work the next morning. Grout is usually injected in a strict primary-secondary pattern. Alternate primary holes are drilled and grouted first followed by the secondary holes. The level of solidification achieved is verified by exhuming grouted soil bulbs, taking samples of the grouted soil and performing unconfined compression tests on the samples.
	Add stone/gravel column (FEMA 2006b)	The potential for lateral spreading can be mitigated by creating a stable mass of material near the open face of site. This can be accomplished by either densifying the layer of potentially liquefiable soil or solidifying the soil to prevent liquefaction. One of the techniques used to achieve this goal is a vibro compaction methods involving the installation of stone columns can be used to densify the potentially liquefiable layer
Roof connection improvement	URM building (FEMA 2006b)	Considerations for cost depend on the number, type and depth of dowels; the difficulty of access; and the extent of finishes that are impacted. Through bolts are usually less expensive than adhesive anchors. Drilling is loud and can be disruptive to occupants. Typically, either the floor or ceiling has to be removed to install the dowels. Thus, it is usually not practical to install dowels in occupied rooms, though the work can be phased by building area so disruption is minimized.
	Wood building (FEMA 2006b)	In new construction, attachment of roof or floor sheathing to shear walls below typically requires nailing through the sheathing into framing below. While this attachment remains the preferred approach, installation of nailing is not possible where roof or floor finishes cannot be removed. Fasteners connecting sheathing to framing should not be overdriven (not break the face of the sheathing). Where overdriving occurs, fastener capacity may be reduced up to 40%.
External wall connection improvement	External curtain wall (FEMA 2012)	The connections must be detailed with sufficient ductility and rotation capacity to prevent failure. Typically, the panels are seated on two bearing connections at either the top or bottom floor and then have “push-pull” connections at the adjacent floor which resist out-of-plane loading but move laterally in the plane of the panel. While both the rocking and sway mechanisms are effective methods of accommodating story drift, mixing mechanisms in adjacent panels may potentially close the joints between units, resulting in a collision between adjacent panels. To avoid this, the joints between panels have to be sized to allow for panel rotation and translation.
	External non-structural masonry wall (FEMA 2007a)	Connections are designed to transfer uplift loads applied to the roof, and the positive and negative loads applied to the exterior bearing walls, down to the foundation and into the ground. The roof covering (and wall covering, if there is one) is also part of the load path. To avoid blow-off, the nonstructural elements must also be adequately attached to the structure. Because some locales have very aggressive atmospheric corrosion (such as areas near oceans), special attention needs to be given to the specification of adequate protection for ferrous metals, or to specify alternative metals such as stainless steel.
Foundation connection improvement	Steel building (FEMA 2006b)	The cost of a column connection to foundation upgrade is not very expensive. However, this technique is not usually performed only by itself. Costs would be small relative to an overall lateral force-resisting system upgrade and a foundation upgrade. Disruption could be minimal since typically, there are no tenants in the basement. Even if the basement were used for tenant access, such as parking, only a few columns would be affected at a time.
	Wood building (FEMA 2006b)	Addition of foundation anchorage in a crawl space with minimum required code vertical clearance is difficult due to very cramped conditions; work areas are often hard to get to, let alone getting tools and supplies and executing work. New temporary access openings and disconnection of HVAC ducting may occasionally be needed to provide access to work. There are no proprietary concerns with this rehabilitation technique, other than the use of proprietary connectors as part of the assemblage.

Retrofit	Option	Description
Building envelope improvement	Flood shields or panels (FEMA 2013d)	Flood shields or panels are watertight structural systems that bridge the openings in walls to prevent the entry of floodwaters. Flood shields work in tandem with waterproof barriers to resist water penetration. Flood shields transfer food-induced forces into the adjacent structural components and, like sealants, can overstress the structural capabilities of the building. A number of vendors make special doors for permanent installation and drop-in panels or barriers that are designed to be watertight and installed as needed for food protection. Heavy food shields for larger openings may require electrical or mechanical systems to move them into place. Designs should include a consideration of the fact that power may not be available following the event and that access through the opening may be necessary to restore power. A maintenance plan is an important part of a flood proofing system that includes food shields. Flood shields, hinges, hoists, latches, channels, and seals should be inspected periodically for damage
Elevated construction	floodwall (FEMA 2013d)	Permanent floodwall is the elevated barriers that provide food protection to one or more buildings. It provides structural protection for shallow food depths; floodwall can provide effective food protection to buildings that experience food levels of 4 feet or greater. A floodwall is a freestanding, permanent, engineered structure designed to prevent encroachment of floodwaters. Existing building foundation conditions may affect the floodwall design directly. For example, evidence of seepage or cracking in foundation walls may indicate the need to relieve hydrostatic pressure on the foundation as part of the floodwall design. Floodwall design depends primarily on the type of flooding expected at the building site. High water levels and velocities can exert significant hydrostatic and hydrodynamic forces that must be accounted for in the floodwall design
Foundation-tie improvement	Adhesive anchors (FEMA 2006b)	For wood building, it is most common to use adhesive anchors for anchorage of the vertical tie-down bolt to the foundation. It has been common to install the adhesive anchor straight down into the footing, or at a very slight angle if required for access. Adhesive anchors must be installed in accordance with the manufacturer's recommendations and the applicable ICC Evaluation Services report recommendations. Tie-down connectors should be attached to substantial existing footings that have the shear and flexural capacity to mobilize required resistance. Alternately, new footings or footing reinforcement can be provided.

Table 3-31. Summary of retrofit solutions for buildings for natural hazards

3.8.4 Life-cycle cost

For essential building infrastructure, life-cycle cost analysis (LCCA) should be considered in plan and design. This topic will be further discussed in Chapter 8.

3.9 Discussion

Multiple natural disasters are common in the subject countries. Large earthquakes happen occasionally, while hurricane and flood occur more frequently. Throughout all stages of disaster, from the event itself to the end, full recovery, buildings like emergency centers, government buildings, hospitals, schools and hotels provide critical aid in response to needs of rescue, in restoring communities, treating injuries and accommodating displaced people. For these essential missions, the structural resiliency of these buildings against multiple disasters is a vital factor. If these types of buildings are not functional due to lack of disaster resistance, and one of the above essential missions cannot be executed, it could induce a secondary disaster that would have otherwise been avoidable. The implementation of resilience measures against natural disasters is then an urgent matter, not only for disaster damage reduction, but also for disaster response.

A summary of the resilience measures, associated cost and vulnerability reduction according to critical hazard and the vulnerable components of buildings is presented in Table 3-32. The improvement cost is an average estimation of the onetime implementation of the resilience measure. Vulnerability reduction was evaluated based on the likelihood of physical damage occurrence; any functional outages or indirect losses were not considered. As seen in the table, the vulnerabilities of critical components are largely reduced by strengthening so much so that the effect of the resilience measure is obvious.

For future feasibility studies, functional and indirect impacts, in addition to physical damage, should be considered for a cost-benefit analysis of building strengthening, especially for critical buildings (i.e., essential facilities) under a natural disaster. It is also recommended that multiple disaster events during an expected building's service lifetime be taken into account, because those buildings are typically used for longer periods than ordinary buildings.

Subsector	**Hazard**	**Vulnerable Component**	**Resilience Measure**	**Cost, % of initial cost (Est.)**	
				Improvement cost	**Vulnerability reduction**
Schools, Hospitals, Government buildings, Emergency centers, Hospitality and hotels	Earthquake shaking	Wall out of plane	Add wall bracing (anchor and strongback)	15%	40%
		In plane	Add RC shear wall or Steel brace	20%	60%
		Structural irregularity	Add Shear wall, Moment frame or Braced frame	15%	40%
		Ductility	Add Column jacketing or Beam retrofit	10%	30%
		Stiffness and capacity	Add Steel brace or Steel plate shear wall	15%	50%
		Energy absorption	Add Seismic isolation or Energy dissipation damper	40%	80%
	Earthquake liquefaction	Foundation	Enlarge Spread footing or Add pile foundation	40%	80%
		Soil	Install Soil grouting or Stone/gravel columns	30%	80%
	Wind	Roof connection	Improve Roof connection anchor	10%	50%
		External curtain wall and wall	Improve Wall connection attachment	10%	30%
		Foundation connection	Add Connection anchor bolts and stiffeners	10%	50%
	Flood	Building envelope	Install flood shield	10%	80%

		Elevated construction	Elevate floodwall	15%	80%
		Foundation tie	Strengthen anchoring to foundation	10%	50%

Table 3-32. Summary of findings for building infrastructure in the Caribbean

4. WATER INFRASTRUCTURE

4.1 Introduction

Improvement in resiliency of water infrastructure (see Figure 4-1) can have a multi-fold beneficial impact for the Caribbean. By reducing physical damage to water infrastructure components, i) economic losses are reduced; ii) secondary impacts, such as flooding of roadways as a result of water pipes bursting, are reduced; and iii) maintaining the delivery of potable ("drinking") water, coupled with treatment and discharge of wastewater, accelerates recovery and allows for operation of critical emergency facilities, such as hospitals and response centers. Water treatment plants (WTP), wastewater treatment plants (WWTP), and underground potable water pipelines (UGP) are critical infrastructure lifelines and will be emphasized in this chapter.

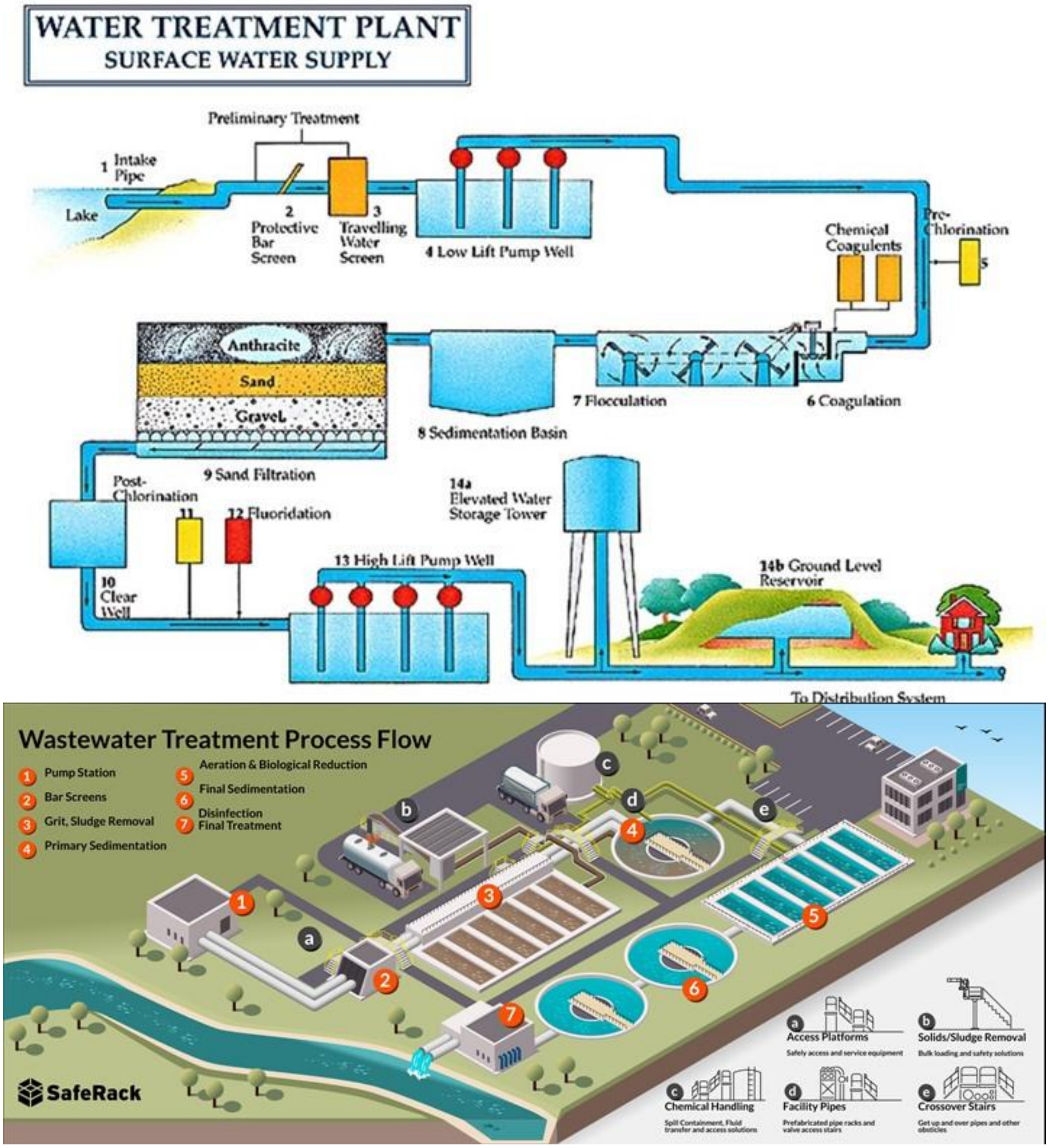


Figure 4-1. Water (and wastewater) infrastructure (T: PMC, 2020; B: SafeRack, 2020)

For water infrastructure in the Caribbean, the hazard and infrastructure components listed in Table 4-1 will be used. As noted in the table, for WTP and WWTP, earthquakes, and to a lesser extent, hurricanes and flooding, are critical, whereas for UGP systems, ground movement and liquefaction are the most critical hazard. It is assumed the discussion presented herein applies to all countries in the Caribbean, as water sector construction is likely similar across the subject countries due to significant cooperation between utility providers.

Sector	Subsector	Type	Component	Hazard
WATER	Water treatment plants Wastewater treatment plants	Plant building (Masonry/RC frame)	Wall	Earthquake
			Ductility	Earthquake
			Roof	Wind
			External wall	Wind
			Opening	Flood
			Elevation	Flood
		Water storage	Foundation	Earthquake
			Anchorage	Earthquake
		Equipment (mechanical and electrical)	Foundation	Earthquake, Wind
			Elevation	Flood
			Flood proofing	Flood
		All	Foundation	Liquefaction
	Underground pipelines	Pipe	Brittle pipes (Asbestos-Cement, Cast Iron, etc.)	Earthquake, liquefaction, flood
		Joints	Conventional	Earthquake, liquefaction

Table 4-1. Analysis matrix for water infrastructure

In this report, the main water pipes (larger diameter) transferring water from WTP to property junctions will be discussed. The secondary pipes (smaller diameter) carrying water to individual structures are not considered. Figure 4-2 shows the distinction between main and secondary pipelines.

Main water pipes Secondary water pipes

Figure 4-2. Main and secondary pipes (various)

4.2 Geographical distribution

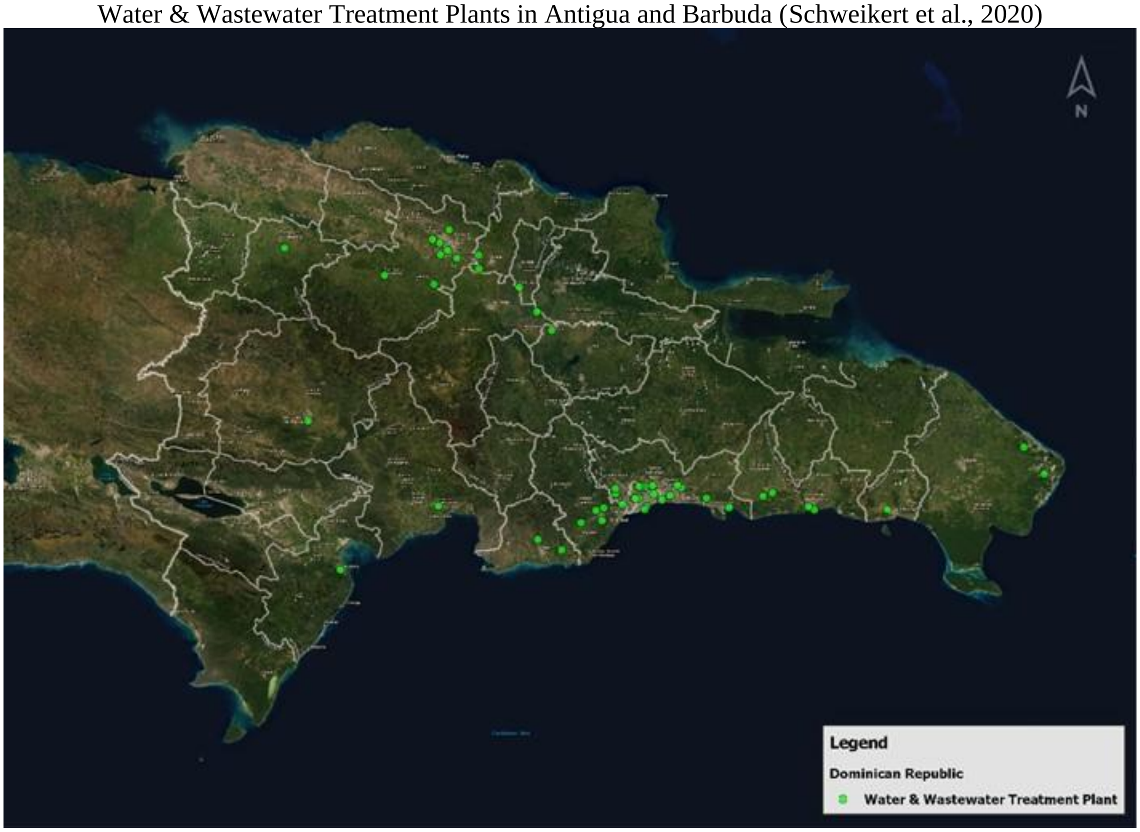
In the subject countries, the following water utility companies are responsible for construction, maintenance, and upgrading of water infrastructure; see Table 4-2.

Country	Utility
Suriname	N.V. Surinaamsche Waterleiding Maatschappij
Trinidad and Tobago	The Water and Sewerage Authority of Trinidad and Tobago (WASA)
Guyana	Guyana Water Incorporated (GWI), Severn Trent Water International (STWI)
Belize	Belize Water Services (BWS)
Haiti	National Directorate for Water Supply and Sanitation in the Ministry of Public Works (Direction Nationale d'Eau Potable et d'Assainissement, DINEPA)
Dominican Republic	Regional companies: the Santo Domingo Water and Sewerage Corporation (CAASD), the Santiago Water and Sewerage Company (CORAASAN), the Puerto Plata Water and Sewerage Company (CORAAPPLATA), the Moca Water and Sewerage Company (CORAMOCA), the Romana Water and Sewerage Company (CORAAROM)
Antigua and Barbuda	The Caribbean Water and Sewerage Association
Dominica	Dominica Water and Sewerage Company Limited (DOWASCO)
Grenada	Grenada's National Water and Sewerage Authority (NAWASA)
Saint Kitts and Nevis	The Water Department works under the Ministry of Public Works
Saint Lucia	The government-owned Water and Sewerage Company
Saint Vincent and the Grenadines	The Central Water and Sewerage Authority
Sint Maarten	NV GEBE
Barbados	Barbados Water Authority Limited
Jamaica	The National Water Commission (NWC)
Bahamas	The Bahamas Water and Sewerage Corporation (WSC)

Table 4-2. Water utility providers for 16 subject countries

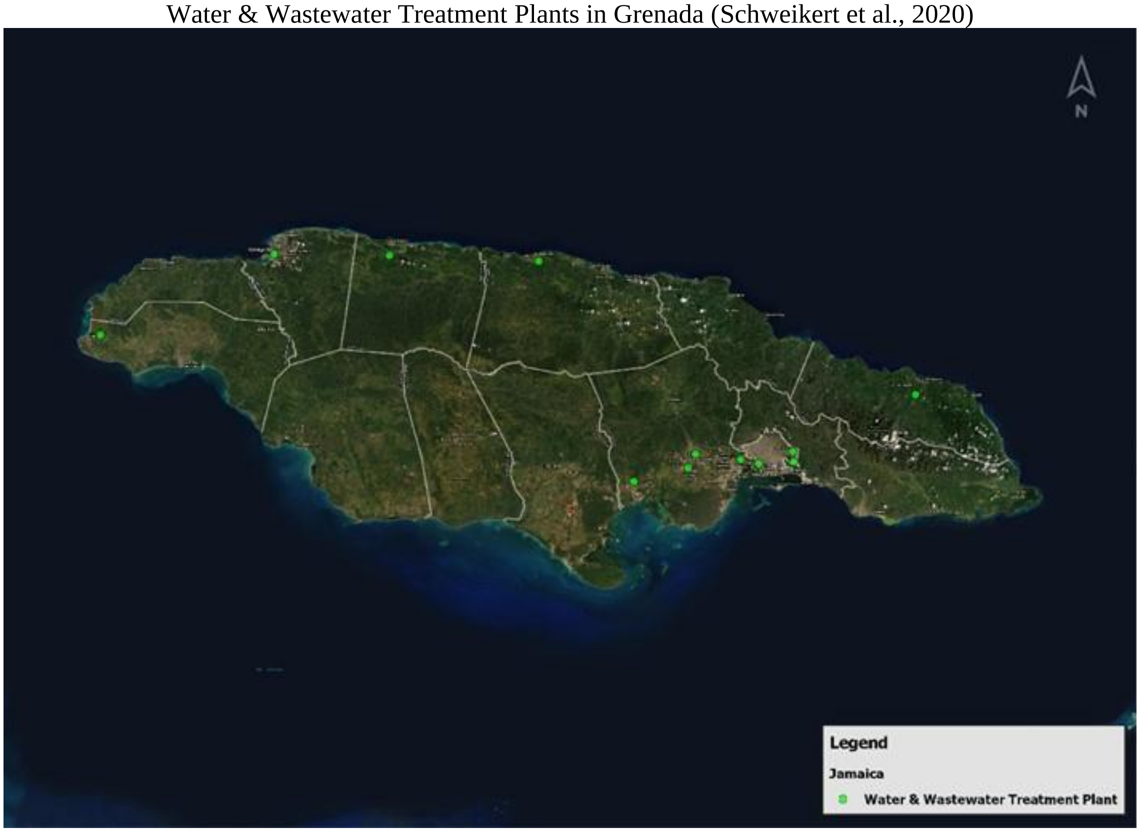
Examples of water distribution maps in the subject countries are presented in Figure 4-3. As discussed previously, the report will focus on water treatment plants (WTP), wastewater treatment plants (WWTP), and underground potable water pipeline (UGP) systems in the Caribbean. Figure 4-4 shows the location of WTP and WWTP in a number of subject countries.

Water & Wastewater Treatment Plants in Antigua and Barbuda (Schweikert et al., 2020)



Water & Wastewater Treatment Plants in the Dominican Republic (Schweikert et al., 2020)

Water & Wastewater Treatment Plants in Grenada (Schweikert et al., 2020)

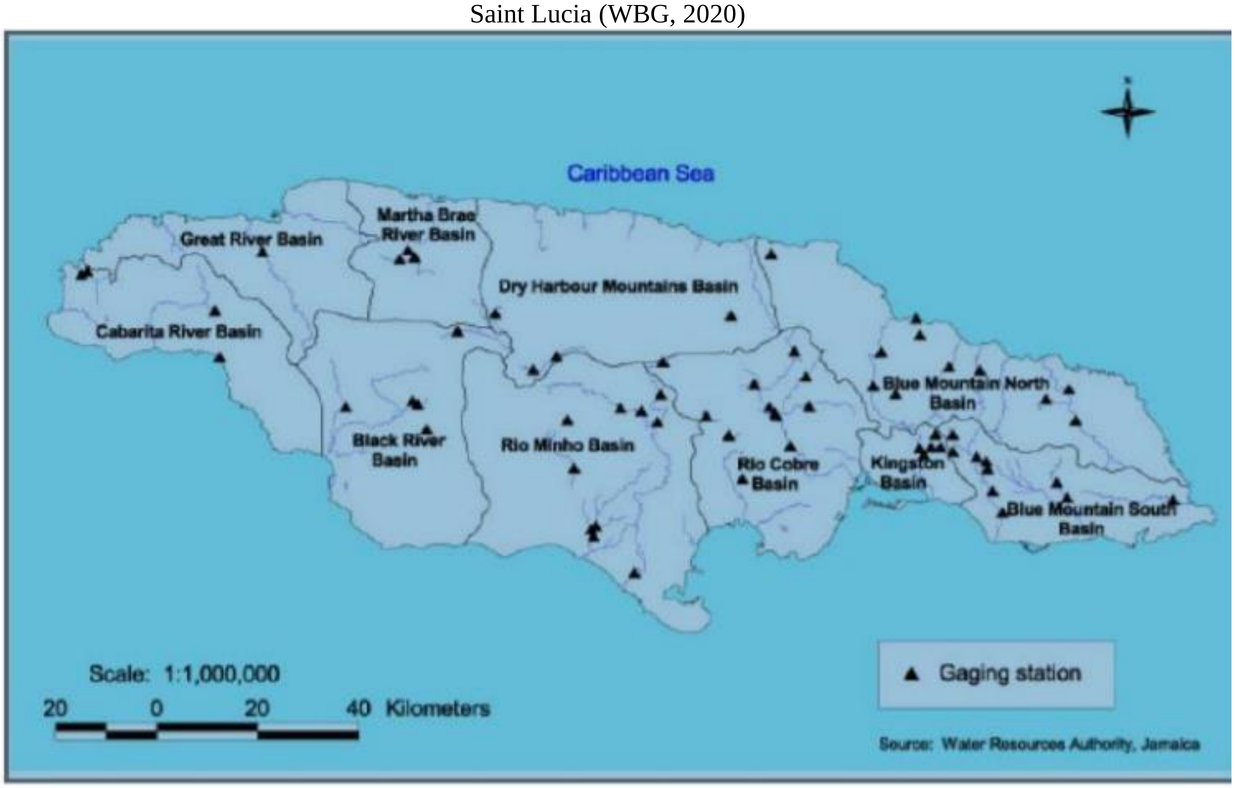


Water & Wastewater Treatment Plants in Jamaica (Schweikert et al., 2020)

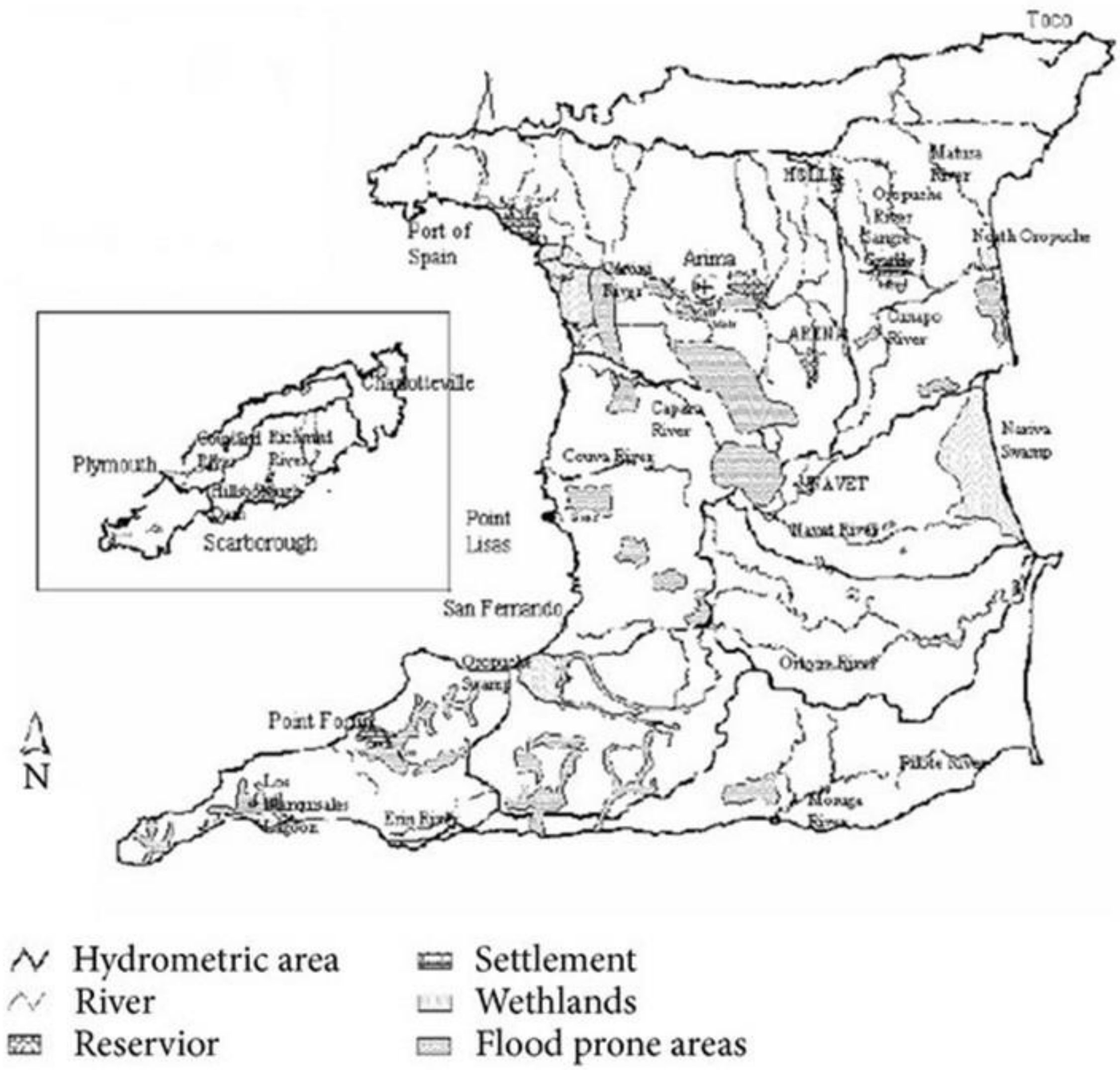
Water & Wastewater Treatment Plants in Trinidad and Tobago (Schweikert et al., 2020)

Water intake locations in Saint Lucia (Gosine, 2020)

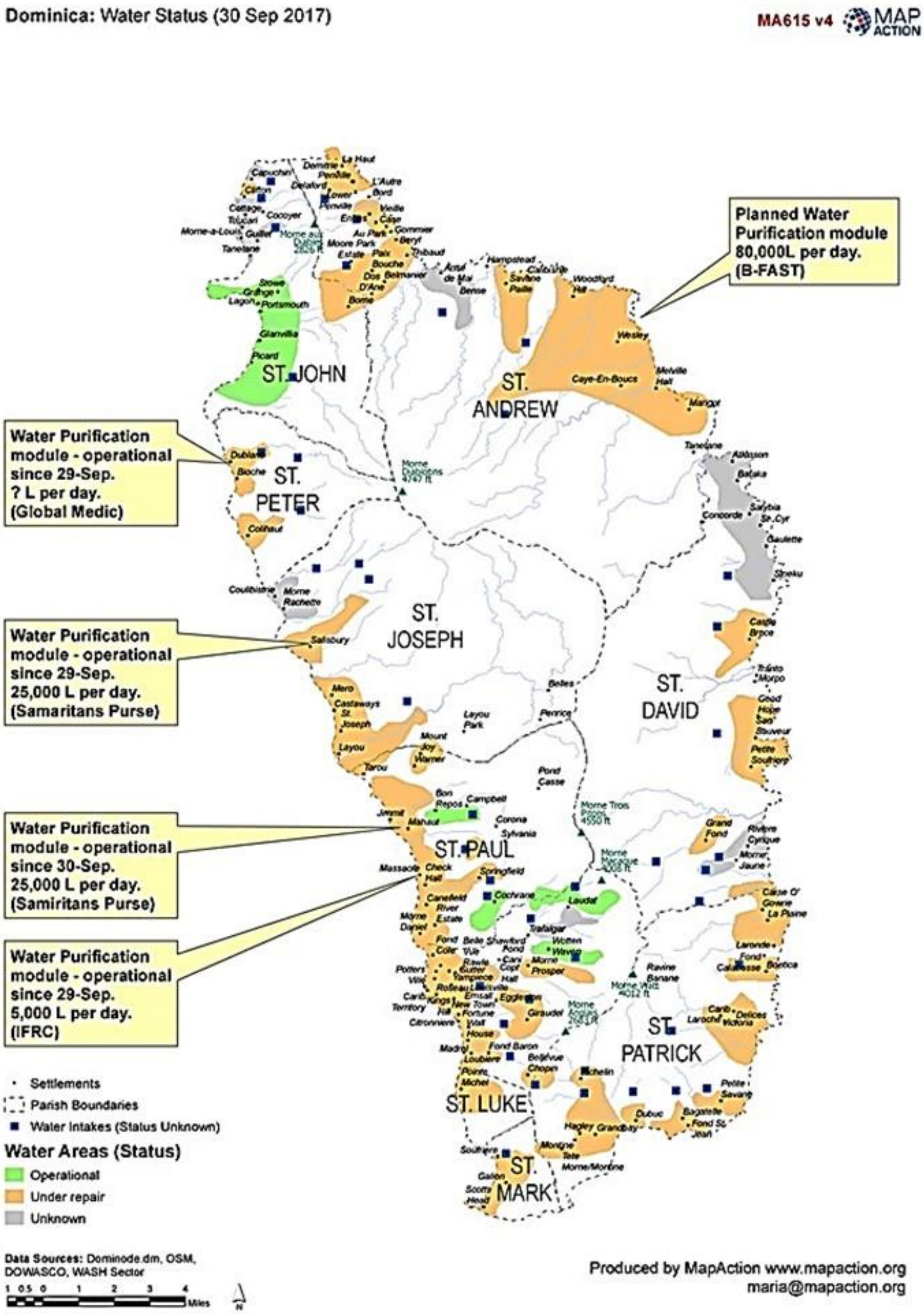
Saint Lucia (WBG, 2020)



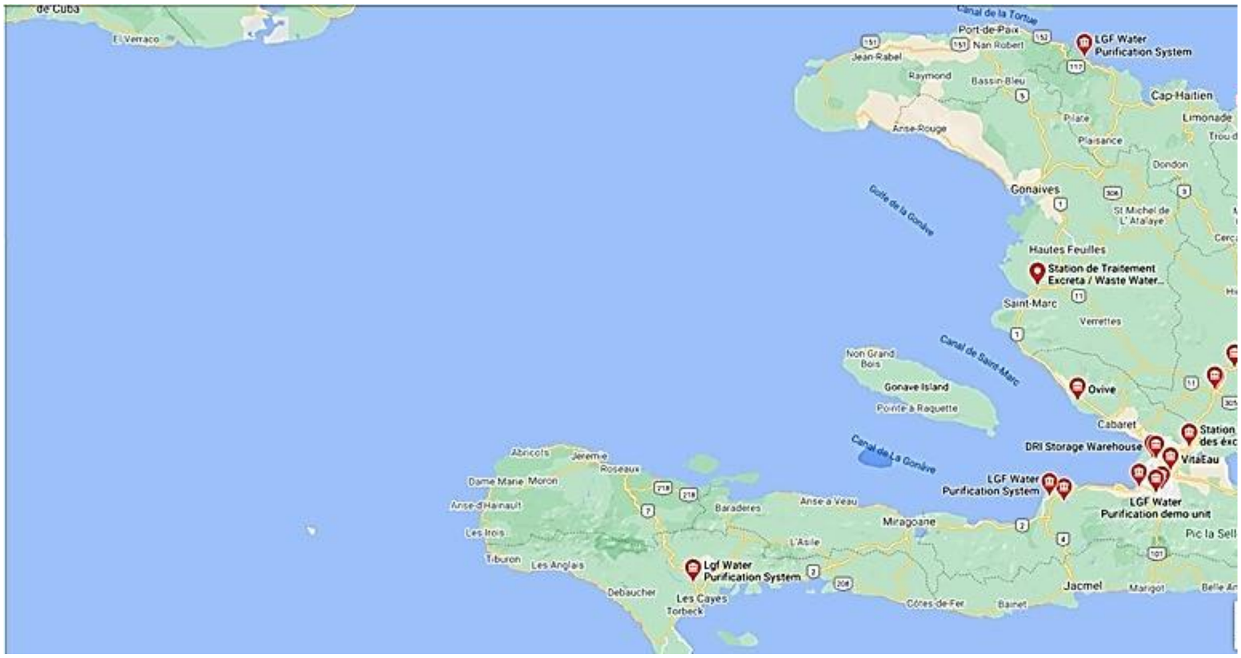
Jamaica (Aquastat, 2015)



Trinidad and Tobago (Tota-Maharaj and Meeroff, 2013)

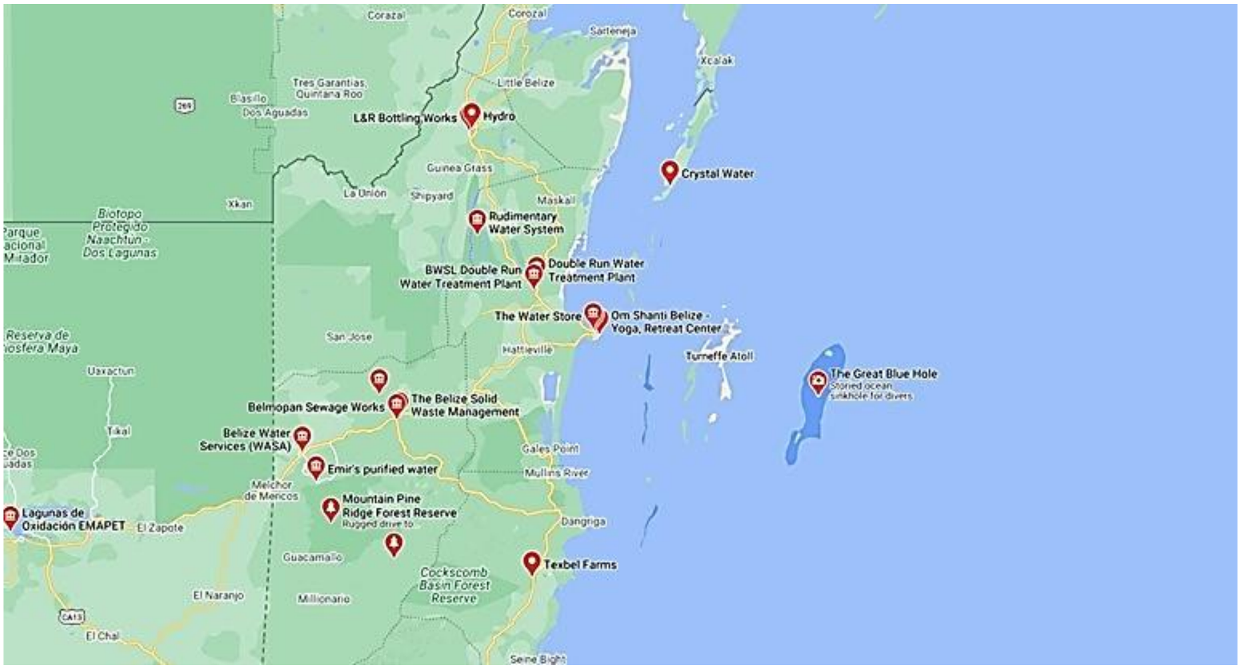


Dominica (Dominica News, 2017)
Figure 4-3. Water distribution networks



Haiti

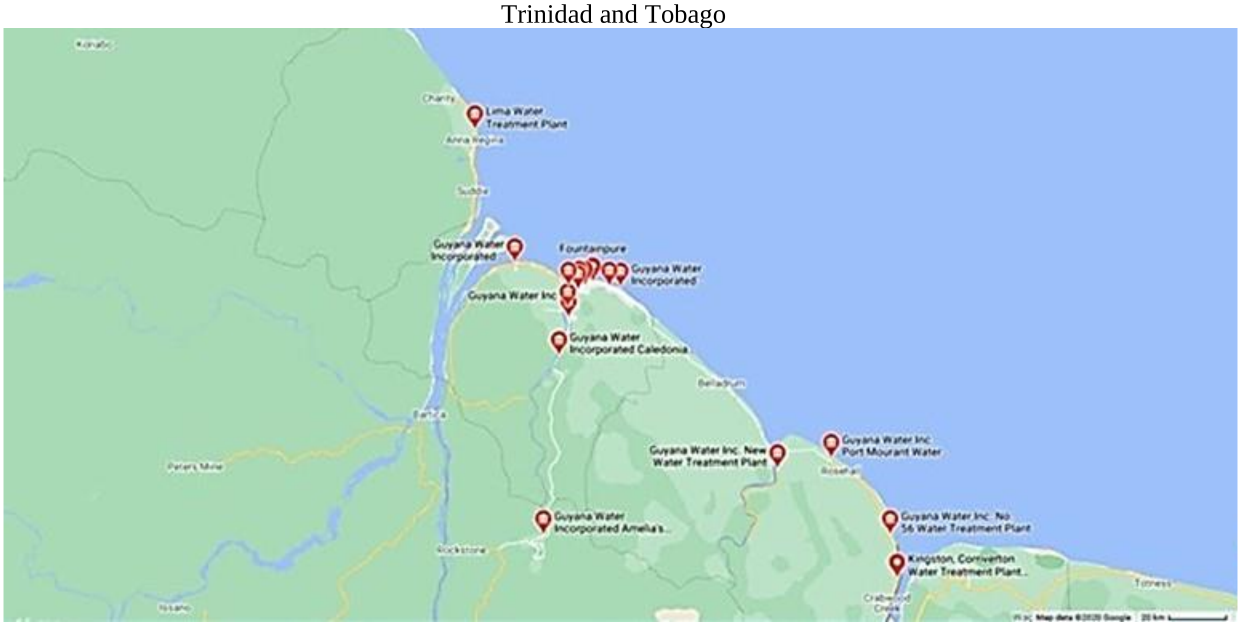
Domincan Republic



Belize

Jamaica

Trinidad and Tobago



Guyana

Suriname

Figure 4-4. Typical locations of WTP and WWTP in subject countries

Table 4-3 presents data regarding the water infrastructure for selected countries. In recent years, upgrades to water sector components in several countries have been undertaken or are planned. Examples include:

- Dominican Republic (Hazen, 2017):

  To provide safe drinking water to the urban service areas of La Romana, San Cristobal, and San Francisco de Macoris, the National Institute of Water and Sewage of the Dominican Republic (!NAPA) retained the Biwater/Sinercon Consortium for the Design-Build delivery of three 87,000 m3/d Water Treatment Plants and associated infrastructure. In turn, Biwater U.S.A. retained Hazen and Sawyer to provide engineering design and support in the procurement management services for the project. The project consisted of building three water treatment plants (The Mata Larga Plant, La Romana Plant and the San Cristobal Plant), to serve approximately 500,000 people, each of which is rated at 1 m3/s (23 mgd) capacity. Other supporting infrastructure development included nine water storage tanks, seven pump stations with capacities of up to 91,000 m3/d, and 130 km of ductile iron raw water and transmission mains, ranging in size from 200 to 1,000 mm in diameter.

- Dominican Republic (Waterworld, 2009):

  The country's national aqueduct and sewerage authority INAPA has awarded ACCIONA's water treatment services arm a 74 million euro contract for the construction and commissioning of a drinking water plant (DWTP) for the Republic's southern Peravia province, with a capacity of 86,400m3/day. The contract also includes the works for a 17.5km adduction line and a water supply network for the surrounding towns and villages. The project deadline is two years.

- Dominica (DOWASCO, 2020):

  The Government of Dominica has secured funding to the tune of twenty-one million dollars from the European Union to undertake a major project to significantly improve the water supply to residents along the west coast of Dominica. The Government itself is also investing a further four million dollars in the project, bringing the total cost to twenty-five million dollars EC. Pipe laying works have been completed along most of the highway between Salisbury to Portsmouth while excavations and pipe laying works commenced from the Cabrits junction towards Capuchin in February 2012 and are expected to continue until June 2012. So far over 20 Km of pipelines have been laid. Lot Three will see the construction of the nine new storage

- Jamaica (NWC, 2020):

  The National Water Commission (NWC) says the supply of water to several communities in the Corporate Area should improve with the new Ferry 600-millimetre diameter pipeline. The new pipeline replaces the previous 18-inch diameter pipeline, popularly known as the Ferry 18-inch. NWC said it serves communities along Washington Boulevard up to Dunrobin Avenue, where it interconnects with the recently commissioned 700-millimetre diameter transmission main that was laid along Constant Spring Road.

- Guyana (GWI, 2020)

  There were several breakages on Vlissengen Road and Church Street, Georgetown. However, the Agency received resources to purchase new HDP Pipes, which will be replaced. In the 2020 Budget, there will be a replacement of the transmission line for Cemetery Road, Georgetown, since the area is a critical artery for the supply of water to the City. Other areas in the City, such as Kitty, Kingston and Albouystown will get lines replacement after the operation is executed at Cemetery Road, Georgetown.

- St Lucia (WASCO, 2020)

  Construction of a reinforced concrete Raw Water Intake in the Tournesse River; Construction of a Water Treatment Facility and pumping station for a minimum production of 0.4 Million Imperial Gallons per Day of drinking water, meeting the World Health Organization requirements, Installation of 2.5km of ductile iron 300mm diameter pipe (buried) from the Raw Water Intake to the Treatment Plant, Installation of 2 km of High Density Polyethylene (HDPE) as transmission pipeline to various communities in DenneryNorth.

Country	Capacity, MGD (Millions of gallons/day)	WTP	WWTP	UGP, km
Guyana			>18	
Antigua and Barbuda	7.0	3 (desalinization), 2		
Dominica	10 (river-based)			
Grenada				100
Saint Kitts and Nevis	2.2			300
Barbados	35		2	3,200
Jamaica	190	1,000	100	4,000
Bahamas				

Table 4-3. Examples of water sector components (various)

4.3 Typology

Examples of WTP in the subject countries are presented in Figure 4-5. Figure 4-6 presents examples of WWTP. Figure 4-7 presents example of pipelines in the subject countries.

Trinidad and Tobago WTP (WASA, 2020)

Trinidad and Tobago WTP (Desalinazation), (WASA, 2020)


Desrruisseaux, (St. Lucia) WTP 860 m3/day WTP (WASCO, 2017)

Building

Equipment

Blue Hills (Bahamas) 45,000 m3 WTP (Consolidated Water, 2020)

Buildings and water tanks

Filter basin

San Cristobal (Dominican Republic) 1 m3/sec WTP (Hazen, 2020)

Saint Vincent and Grenadine WTP (CWSA, 2020)

Sint Maarten desalinization plant (SMN-N 2012)

Antigua WTP (Desalinazation), (APUA, 2020)

Figure 4-5. WTP in subject countries

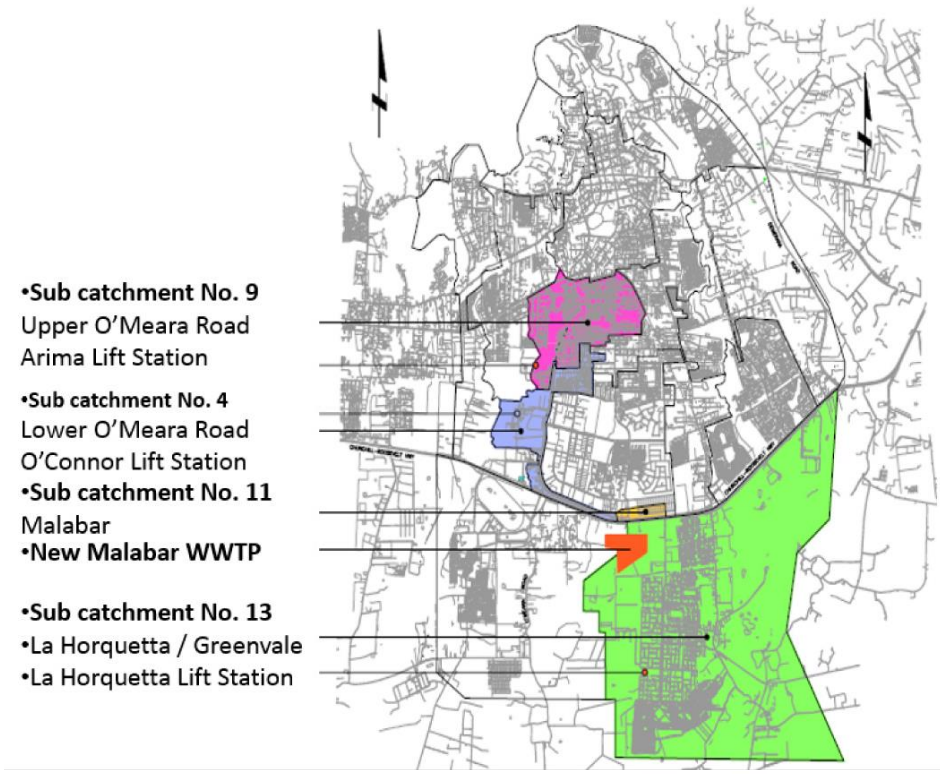
Sedimentation pond

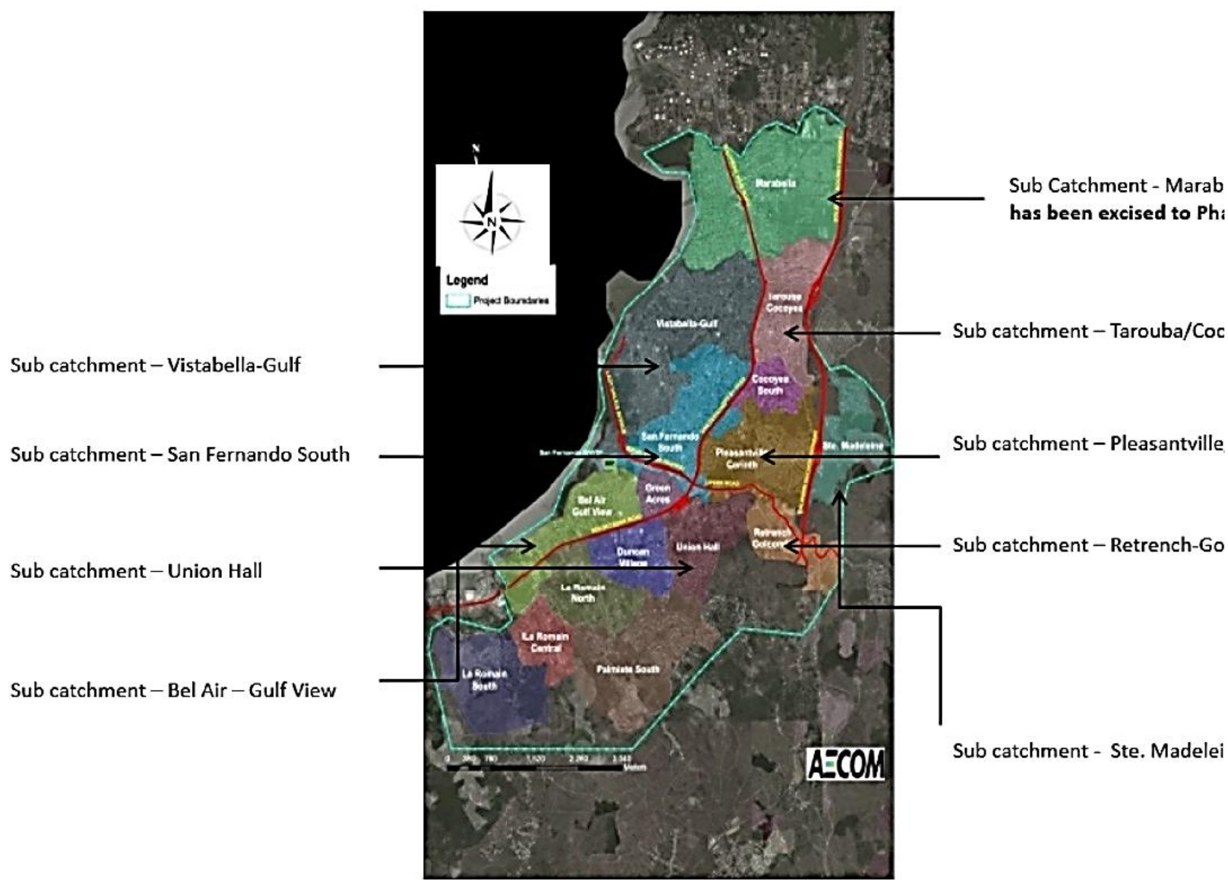
Soapberry (Jamaica) 60 MGD WWTP (various)

Structure

Facility

Bridgeport and South Coast WWTP (Barbados) (BWA, 2020)

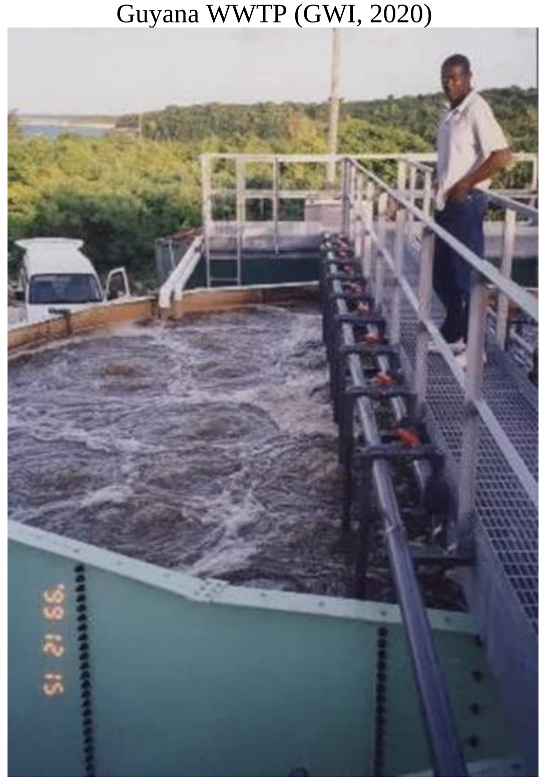




Trinidad and Tobago WWTP (WASA, 2020)

Saint Vincent and Grenadine WWTP (CWSA, 2020)

Guyana WWTP (GWI, 2020)



Antigua

Antigua WWTP (APUA, 2020)

Figure 4-6. WWTP in subject countries

2-m concrete pipe; Dominican Republic (Hazen, 2020)

0.6-m pipe Jamaica (NWC, 2020)

0.15 m Haiti

Saint Kitts and Nevis

0.6 m St Luicia

0.6 m pipe Bahamas

(Various)

0.3 pipe Saint Vincent and the Grenadines (CWSA, 2020)


0.35-m pipe Guyana (GWI, 2020)

Figure 4-7. Water pipelines in subject countries

4.4 Vulnerable components

Past natural hazards have shown that water infrastructure components are vulnerable to various natural hazards. Certain components have been known to experience frequent failures. Strengthening of these components would lead to marked improvement in the performance and as such, these components are emphasized in this chapter. The damage assessment conducted by the National Water and Sewerage Authority (NAWASA) in the aftermath of Hurricane Ivan in 2004 is presented in Table 4-4 (OECS, 2004). Table 4-5 and Table 4-6 present examples of damage to water infrastructure in Dominica as a result of recent hurricanes and ensuing flooding. Table 4-7 lists the damage to water infrastructure in Saint Vincent and the Grenadines due to flooding.

Components	US$M
Buildings	1.0
Water plants	0.6
Pipelines	1.0
Total	**2.6**

Table 4-4. Damage to Grenada’s water infrastructure, Hurricane Ivan (OECS, 2004)

Loss components	US$M
Damage	14.5
Losses	2.3
Total	**16.8**

Table 4-5. Damage to Dominica’s water infrastructure, Hurricane Erika (ACP-EU, 2015)

Subsector	Components	US$M
Water supply	Production and distribution	19.9
Sanitation	List stations, mains, connections	3.6

Table 4-6. Damage to Dominica's water infrastructure, Hurricane Maria (ACP-EU, 2017)

Loss components	US$M
Damage	0.25
Losses	0.03
Total	**0.28**

Table 4-7. Damage to Saint Vincent and the Grenadines water infrastructure, 2016 floods (ACP-EU, 2016)

Water infrastructure is vulnerable to damage from earthquakes; see Figure 4-8. Examples of natural damage to vulnerable water infrastructure components in the subject countries are presented in Figure 4-9.

WTP

WTP

Calexico WWTP


WWTP


Pipelines

Figure 4-8. Water infrastructure (WTP and WWTP) damage, earthquake hazard (EERI, 2010)

Damaged water infrastructure - Dominica

Broken pipeline - Haiti

Dominica

Guyana

Figure 4-9. Damage to pipelines (various)

Table 4-8 presents the components that have experienced the most damage in past events and will be considered in this chapter.

Subsector	Type	Component	Vulnerability	Hazard
Water treatment plant	Plant building (Masonry/RC frame)	Wall	Out-of-plane capacity	Earthquake
			In-plane capacity	Earthquake
		Ductility	Non-ductile detailing	Earthquake
Wastewater treatment plant		Roof	Roof connection	Wind
		External wall	External wall	Wind

		Opening	Building envelope	Flood
		Elevation	Elevated construction	Flood
	Water storage	Foundation	Foundation	Earthquake
		Anchorage	Anchorage	Earthquake
	Equipment (mechanical and electrical)	Foundation	Foundation tie	Earthquake, Wind
		Elevation	Elevating equipment	Flood
		Floodproofing	Dry floodproofing	Flood
	All	Foundation	Inadequate capacity	Liquefaction
Pipelines	Pipe	Brittle pipes (asbestos-cement, cast iron, etc.)	Brittle pipes	Earthquake, Liquefaction, Flood
	Joints	Conventional	Limited deformation capacity	Earthquake, liquefaction

Table 4-8. Vulnerable components of water infrastructure and corresponding hazards

4.5 Design practice

4.5.1 Overview

It is anticipated that the buildings and equipment for WTP/WWTP in the Caribbean are designed and constructed by utilizing international design codes or the Caribbean building codes discussed in Section 3.5 because design codes specialized for WTP/WWTP structural design cannot be found in the Caribbean countries. In fact, a description of WWTP in Barbados mentions that the plant was designed abroad by a foreign consultant (UNEP, 1997); likewise, information for WTP/WWTP construction in Suriname states that the construction of water supply facilities should refer to the country codes in addition to the codes of the U.S. or Europe (WHO, 1987). The only design guideline for WTP/WWTP is found in Trinidad and Tobago, and it is introduced in Section 4.5.5. In this guideline, similar to the above descriptions, the structural design of WTP/WWTP buildings basically refers to the building code used in Trinidad and Tobago. Therefore, in the following subsections, the design approaches specified in local building codes (see also Section 3.5) and the U.S. codes for water pipelines are briefly summarized.

4.5.2 Earthquake design

In the Caribbean Uniform Building Code, CUBiC (1985), the seismic design methodology for buildings in the Caribbean is specified and could be applied to the design of WTP/WWTP buildings. The design earthquake force is prescribed according to regional seismicity, ductility factor, occupancy importance factor, vibration characteristic and building mass. The structural elements of buildings are then designed to meet the code requirements to resist the earthquake force. For WTP/WWTP plants, the highest class of occupancy importance factor (i.e., 1.5; see Table 3-6) should be taken and then be designed by the larger seismic design force (i.e., 50% larger than a normal building) for better performance in the aftermath of an earthquake. For water pipelines and equipment (e.g., mechanical facility, nonstructural components, etc.), a seismic guideline for water pipelines in the U.S. (ALA, 2005) and a U.S. guideline for reducing seismic damage of nonstructural components (FEMA, 2012) could be utilized for designing those in the Caribbean, respectively.

4.5.3 Wind design

The wind design for buildings is established for the Caribbean countries in CUBiC (1985). In the simplified procedure of three wind design methodologies, the design wind pressure can be calculated by the reference wind velocity pressure, exposure factor, aerodynamic shape factor and dynamic response factor (see Section 3.5.2.1.1). The reference wind velocity is based on a 50-year return period and is based on country wind hazard intensity. The force, instability, deflection and fatigue of building elements are then estimated and designed by applying the design wind force. In the wind design component of CUBiC, any kind of occupancy importance factor cannot be found. For water pipelines, wind hazard essentially does not adversely affect the pipelines. For mechanical facilities of WTP/WWTP plants, the large overturning, excessive deflection and base anchoring are major concerns caused by wind hazard. The design procedure on nonstructural components of ASCE 7-16 (ASCE, 2017a) could be applied in the Caribbean.

4.5.4 Flood design

As described in Section 3.5.3, the design approach for flood prescribed in ASCE 7-16 (ASCE, 2017a) could be applied for the Caribbean because the International Building Code (ICC 2017 – widely used in the Caribbean) and ASCE 24-14 (ASCE 2015 – document focusing on flood design) refer to ASCE 7-16. Since the base flood for building design is the flood having a 1% chance of being equaled or exceeded in any given year in the code, the Caribbean-specific flood hazard needs to be verified with the base flood prescribed in ASCE 7-16. For flood protection of mechanical facilities and access roads, the 100-year flood elevation or maximum flood of record is usually used to confirm the safety of facilities (PRHEM, 2012).

4.5.5 WTP/WWTP design guideline manual in Trinidad and Tobago (WASA, 2008)

4.5.5.1 Acts, codes and standards

In a section of the Water and Wastewater Design Guideline Manual in Trinidad and Tobago (WASA, 2008), compliance with regulations, acts, codes, standards and guidelines is specified for the design of water infrastructure. The main regulations are summarized below.

- Environmental Management Authority (EMA) of Trinidad and Tobago
- Water And Sewerage Authority (WASA) of Trinidad and Tobago
- Ministry of Public Utilities and Environment (MoPUE)
- National Building Code
- National Fire Code
- OSHA (Occupational Safety and Health Act)
- Trinidad and Tobago Standards (TTBS)
- International Standards (ISO)
- North American Standards (ANSI/AWWA)
- British Standards (BS)

4.5.5.2 Structural design considerations

It is assumed that the design details for water facilities should refer to the National Building Code in Trinidad and Tobago, but general descriptions for the structural design of treatment plants are established in this document as follows:

- The plant shall be sufficient to accommodate anticipated ground moments, including any active geologic faults;
- The designs must assure that the structures are to sustain regional earthquakes and hurricane events;
- The seismic design of civil structures will be generally based on an earthquake zone 3, as defined in UBC 1997 (Uniform Building Code);
- For flood hazard, all WASA buildings or structures shall be flood-proof, and the site for the new facility shall be appropriately selected or designed to be above the 20-year recurrence interval flood line.

4.5.5.3 Construction requirements

The requirements for construction materials are also specified in the design guideline and are summarized below.

- Concrete: the minimum compressive strength of concrete, based on 28 days compressive strength as determined by tests on concrete cylinders according to ASTM International (ASTM), is listed in Table 4-9.

- Steel reinforcement: the reinforcement bars shall be deformed, new steel bars complying with the “Specifications for Billet Steel Bars for Concrete Reinforcement” ASTM A615 Grade 60 or approved equal, with a minimum yield strength of fy = 400 MPa (i.e., 60 ksi).
- Structural steel: the structural steel shall conform with international standards and with the following ASTM standards.
  - ASTM A369 A36M – Specifications for Structural Steel
  - ASTM – A193A & 193M – Specifications for Alloy-Steel and Stainless Steel for Bolting Material for High temperature service.
  - ASTM – A307, A325 & A490 – Specifications for Structural Steel Bolts and Bolted Joints.

Type of construction	Concrete compressive strength	
	(N/mm^2)	**(psi)**
Unreinforced concrete, Lean concrete	10	1,450
Encasement, Duct banks, Cast in place, Concrete curbs	25	3,600
Super structures	35	5,000
Sub-structures	35	5,000

Table 4-9. Required compressive strength for concrete construction

4.6 Fragility functions

4.6.1 Earthquake fragility functions based on FEMA Hazus methodology

4.6.1.1 Overview

In this section, physical (structural) damage fragility relations for WTP, WWTP, and UGP types common in the Caribbean are presented. For the structures, two types of design are considered: **conventional** (non-resilient design constructed prior to adoption of seismic codes) and **enhanced** (resilient construction).

4.6.1.2 WTP

4.6.1.2.1 Damage states

FEMA Hazus (FEMA, 2003a) defines a number of damage states for WTP. The damage states relevant to the Caribbean are summarized in Table 4-10.

Damage state	State	Description
DS1	None	No observable damage
DS2	Slight	Malfunction of plant for a short time (less than three days) due to loss of electric power and backup power if any, considerable damage to various equipment, light damage to sedimentation basins, light damage to chlorination tanks, or light damage to chemical tanks. Loss of water quality may occur
DS3	Moderate	Malfunction of plant for about a week due to loss of electric power and backup power if any, extensive damage to various equipment, considerable damage to sedimentation basins, considerable damage to chlorination tanks with no loss of contents, or considerable damage to chemical tanks. Loss of water quality is imminent.
DS4	Extensive	Pipes connecting the different basins and chemical units being extensively damaged. This type of damage will likely result in the shutdown of the plant.
DS5	Complete	Complete failure of all piping, or extensive damage to the filter gallery.

Table 4-10. WTP damage states (adapted from FEMA, 2003a)

4.6.1.2.2 Fragility functions for ground shaking

Based on whether a WTP is constructed to meet seismic requirements, FEMA Hazus (FEMA, 2003a) has developed fragility functions for WTP corresponding to the probability of exceeding a damage state. Fragility functions relevant to common WTP in the Caribbean are summarized in Table 4-11.

	Mean, PGA, g				Standard deviation, g
Typology	**DS2**	**DS3**	**DS4**	**DS5**	
Stand components	0.16	0.27	0.53	0.83	0.4-0.6
Seismic components	0.2	0.38	0.53	0.83	0.5-0.6

Table 4-11 WTP fragility parameters, ground motion (adapted from FEMA, 2003a)

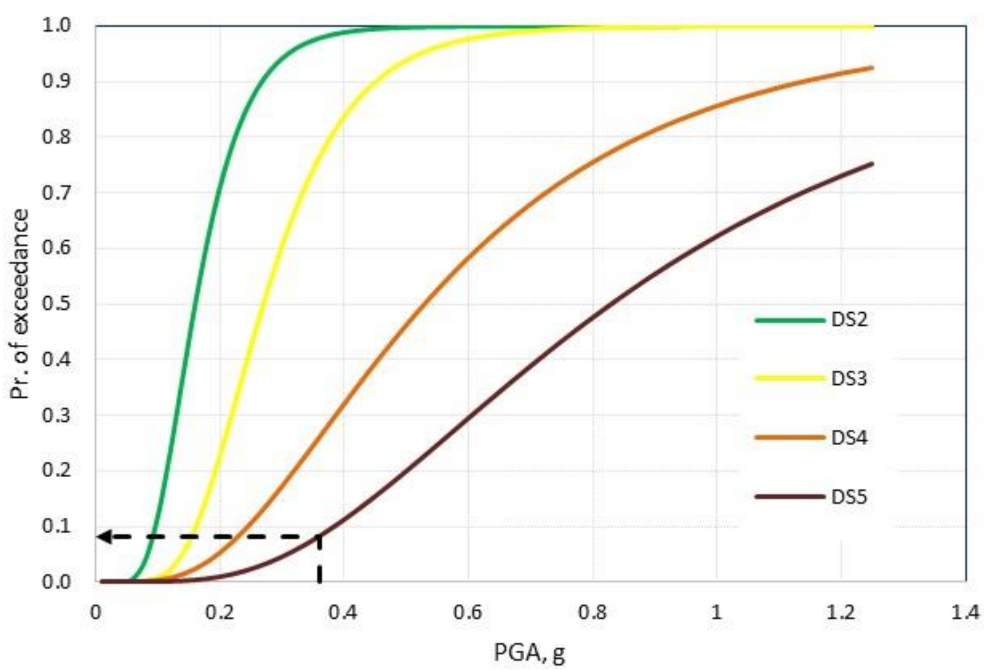
4.6.1.2.3 Fragility functions: ground failure (liquefaction)

The fragility functions for ground failure hazard are similar to what was presented for buildings in Chapter 3.

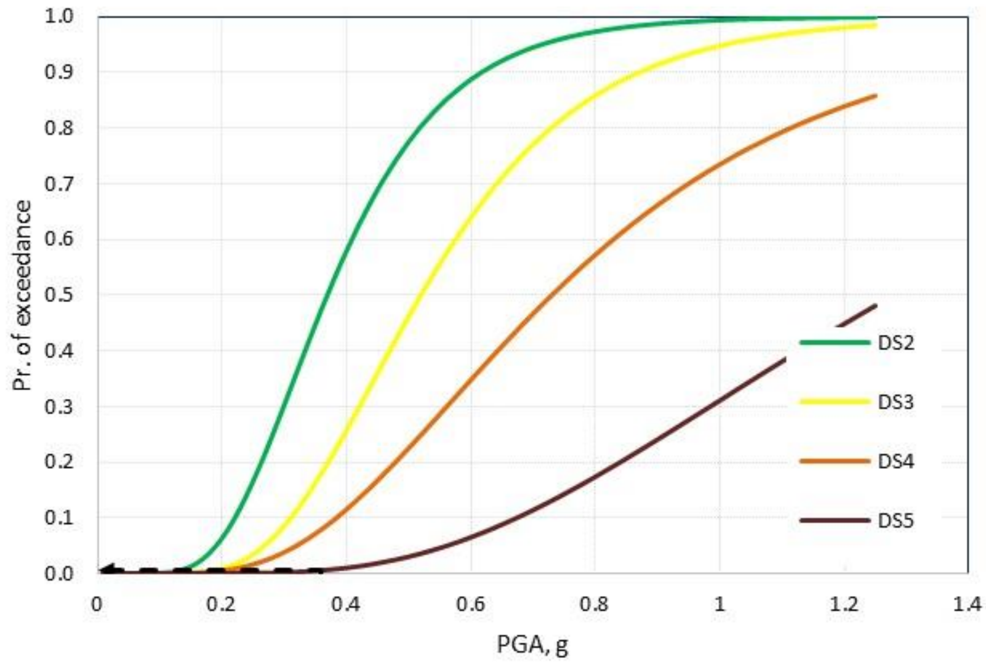
4.6.1.2.4 Typical WTP

Most WTP in the subject countries can be considered as small (10-50 MGD) or medium (50-200 MGD)-sized; see Table 4-3 The larger WTP usually have better engineering and construction standards and thus are more resilient. In this section, it is assumed that existing WTP are based on lower engineering standards and inadequate anchorage, whereas retrofitted WTP are based on better engineering and adequate anchorage of components. Figure 4-10 presents the fragility functions for two WTP: one with conventional design, and one with properly designed anchorage and seismic components. Also shown in the figure is the line with ordinates of approximately 0.36 g, which corresponds to a moderate earthquake. As seen in the figures, the probability of experiencing significant damage is reduced measurably when retrofit measures are implemented.

Figure 4-11 presents the probability of the WTP being in a given damage state when subjected to a moderate earthquake. Note that once seismic retrofitting is implemented, the probability of the WTP experiencing the detrimental DS5 state is reduced from approximately 8% to approximately 1%.



Existing



Retrofitted

Figure 4-10. Fragility functions for example WTP

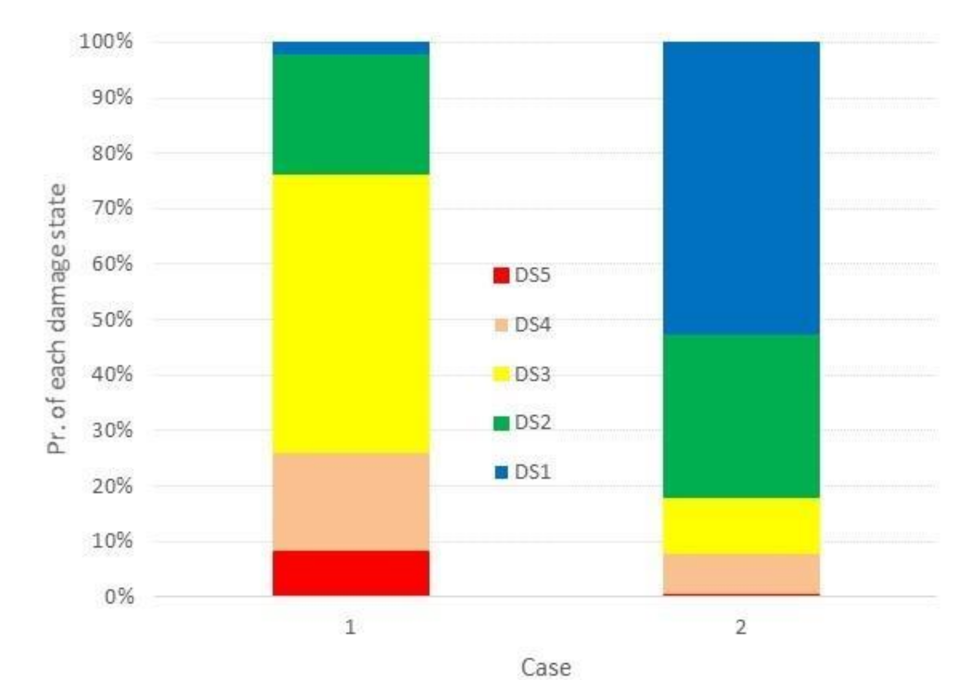


Figure 4-11. Distribution of damage states for WTP

4.6.1.3 WWTP

4.6.1.3.1 Damage states

FEMA Hazus (FEMA, 2003a) defines a number of damage states for WWTP. The damage states relevant to the Caribbean are summarized in Table 4-12.

Damage state	State	Description
DS1	None	No observable damage
DS2	Slight	Malfunction of plant for a short time (less than three days) due to loss of electric power and backup power if any, considerable damage to various equipment, light damage to sedimentation basins, light damage to chlorination tanks, or light damage to chemical tanks.
DS3	Moderate	Malfunction of plant for about a week due to loss of electric power and backup power if any, extensive damage to various equipment, considerable damage to sedimentation basins, considerable damage to chlorination tanks with no loss of contents, or considerable damage to chemical tanks.
DS4	Extensive	Pipes connecting the different basins and chemical units being extensively damaged. This type of damage will likely result in the shutdown of the plant.
DS5	Complete	Complete failure of all piping, or extensive damage to the filter gallery.

Table 4-12. WWTP damage states (adapted from FEMA, 2003a)

4.6.1.3.2 Fragility functions for ground shaking

Based on whether a WWTP is constructed to meet seismic requirements, FEMA Hazus (FEMA, 2003a) has developed fragility functions for WWTP corresponding to the probability of exceeding a damage state. Fragility functions relevant to common WWTP in the Caribbean are summarized in Table 4-13

	Mean, PGA, g				Standard deviation, g
Typology	**DS2**	**DS3**	**DS4**	**DS5**	
Stand components	0.16	0.26	0.48	0.8	0.4-0.55
Seismic components	0.33	0.49	0.70	1.23	0.4-0.55

Table 4-13. WWTP fragility parameters, ground motion (adapted from FEMA, 2003a)

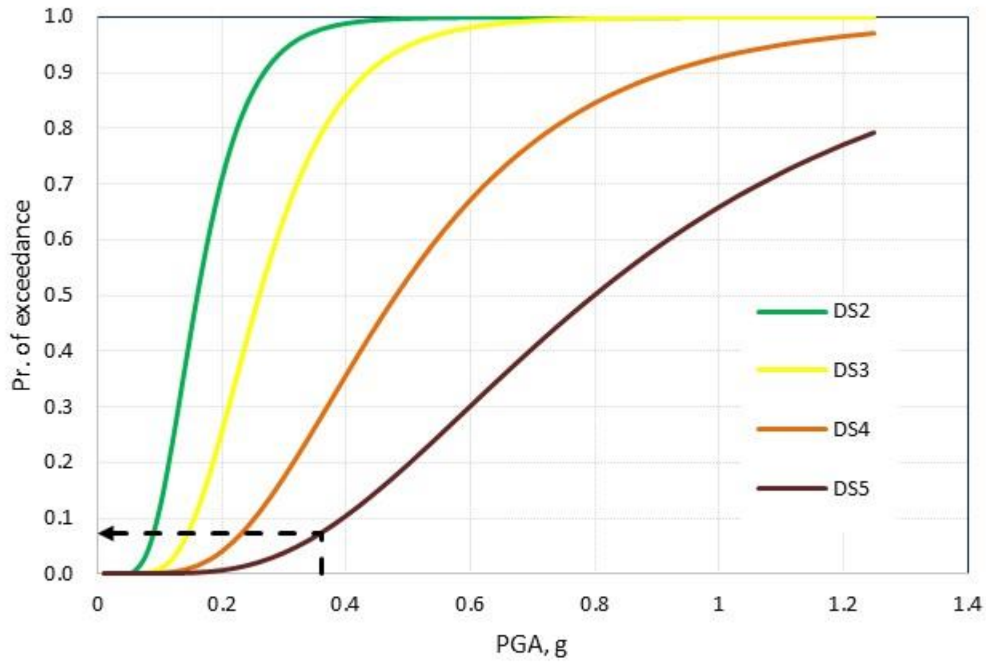
4.6.1.3.3 Fragility functions: ground failure (liquefaction)

The fragility functions for ground failure hazard are similar to what was presented for buildings in Chapter 3.

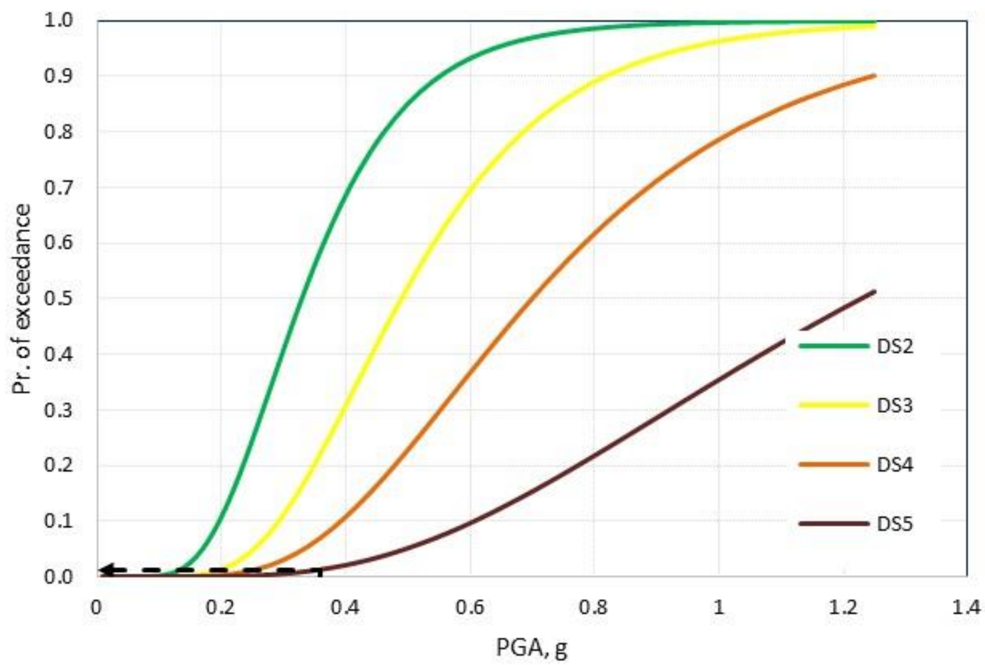
4.6.1.3.4 Typical WWTP

Most WWTP in the subject countries can be considered as small (10-50 MGD) or medium (50-200 MGD)-sized. The larger WTP usually have better engineering and construction standards and thus are more resilient. In this section, it is assumed that existing WWTP are based on lower engineering standards and inadequate anchorage, whereas retrofitted WWTP are based on better engineering and adequate anchorage of components. Figure 4-12 presents the fragility functions for two WWTP: one with conventional design and one with properly designed anchorage and seismic components. Also shown in the figure is the line with ordinates of approximately 0.36 g, which corresponds to a moderate earthquake. As seen in the figures, the probability of experiencing significant damage is reduced measurably when retrofit measures are implemented.

Figure 4-13 presents the probability of the WWTP being in a given damage state when subjected to a moderate earthquake. Note that once seismic retrofitting is implemented, the probability of the WWTP experiencing the detrimental DS5 state is reduced from approximately 7% to approximately 1%.



Existing



Retrofitted

Figure 4-12. Fragility functions for example WWTP

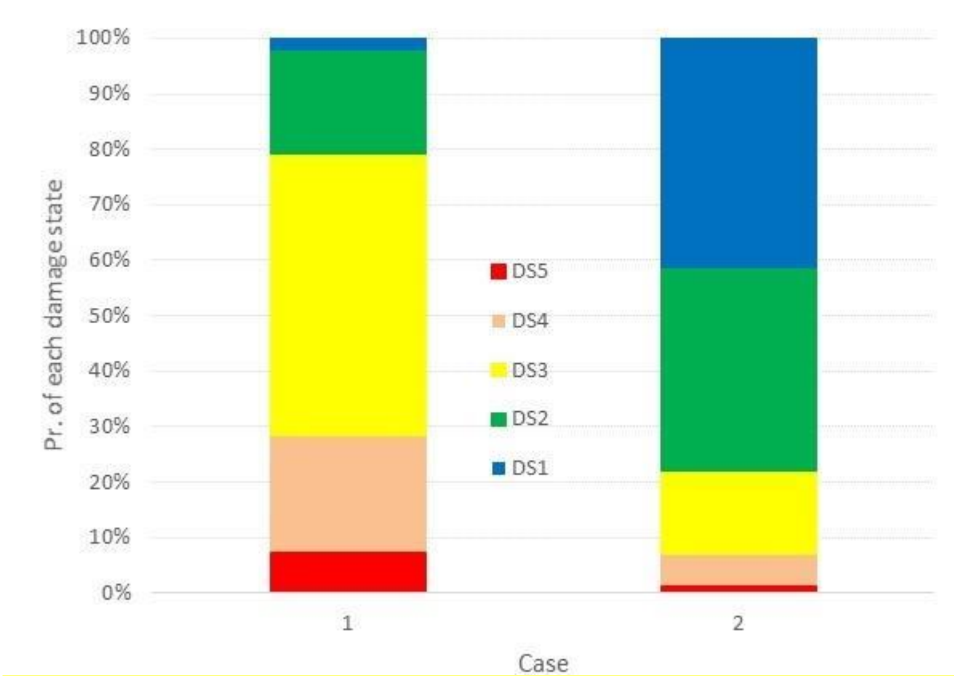


Figure 4-13. Distribution of damage states for WWTP

4.6.1.4 Underground potable water pipes

4.6.1.4.1 Damage states

FEMA Hazus (FEMA, 2003a) defines pipe fragility based on the number of repairs (leaks or breaks) per km of pipe. Fragility is defined based on either peak ground velocity (PGV) or permanent ground deformation (PGD).

4.6.1.4.2 Fragility functions for ground shaking

Pipes are classified as either brittle or ductile; see Table 4-14. Based on whether brittle or ductile pipes are used, FEMA Hazus (FEMA, 2003a) has developed fragility functions for failure (leak or break).

Typology	Examples
Brittle	Asbestos concrete (AC) Cast iron (CI) Concrete (RCC)
Ductile	Ductile iron (DI) Welded steel (WS) High-density polyethylene (HDPE) Polyvinyl chloride (PVC)

Table 4-14. Underground water pipe types (adapted from FEMA, 2003a)

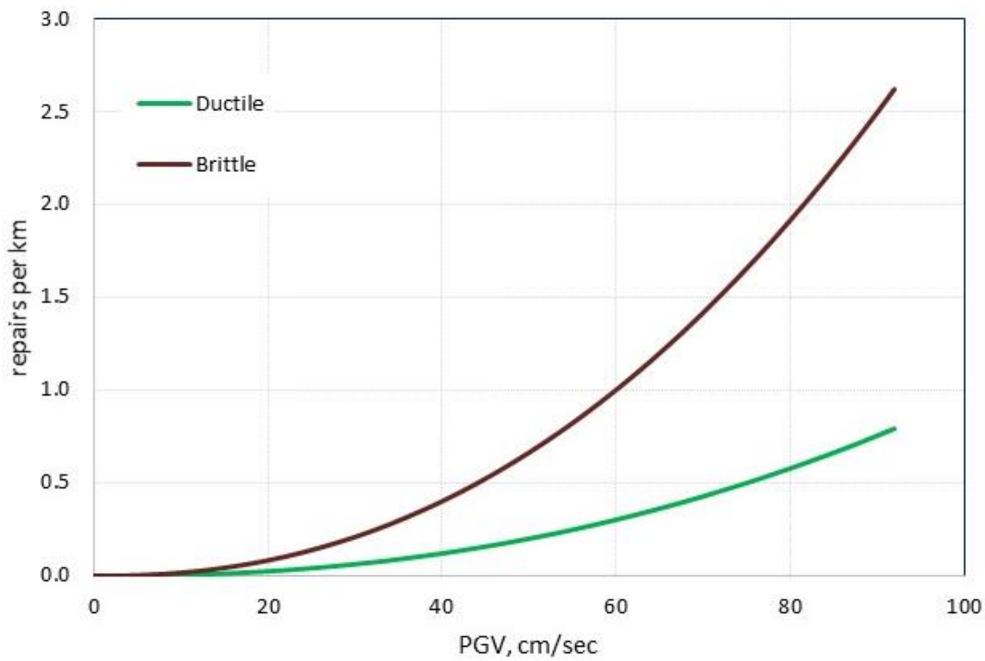
4.6.1.4.3 Fragility functions: ground failure (liquefaction)

The fragility functions for ground failure hazard are similar to ground shaking and are based on PGD.

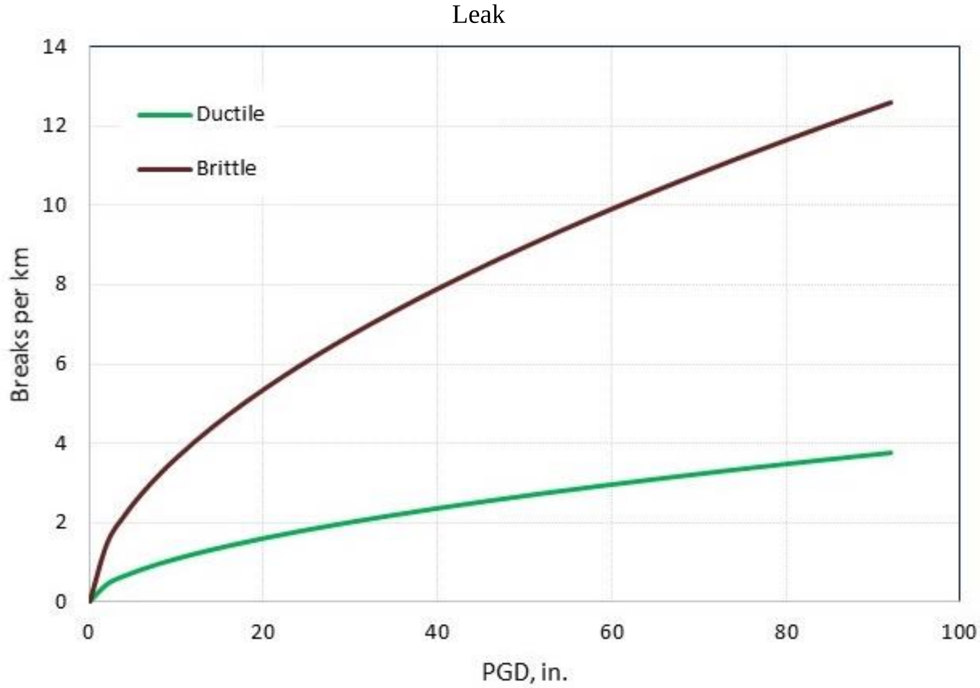
4.6.1.4.4 Typical UGP

Older underground pipes are likely brittle; newer replacement pipes are ductile (see Figure 4-7). Figure 4-14 presents the repair rate (repairs per km) for both leaks and breaks and for brittle and ductile pipes. Note the following:

- For an earthquake with a PGV of 50 cm/sec, the number of leaks in an 80-km pipe is reduced from 52 to 16 when ductile pipes are used. This implies significant savings in both repair cost and service interruption.
- For an earthquake (or liquefaction) with a PGD of 30 in., the number of breaks is reduced from 4 to 1 per km when ductile pipes are used.
- To increase the resiliency of pipe systems, either ductile pipes can be used or alternatively, existing pipes can be used with ductile connections or ductile liners to achieve the same benefits. The latter approach is more cost effective and can be implemented to reduce service interruption times.



Leak



Ductile

Figure 4-14. Fragility functions for example UGP

4.6.2 Wind fragility functions

Wind fragility functions for WTP and WWTP are similar to industrial buildings. UGP are not susceptible to damage from wind loading due to their underground location. Figure 4-15 presents the wind fragility functions for unreinforced (URM) and reinforced masonry (RM) buildings. Note that at a hurricane speed of 200 km/hr (typical in the subject countries), the damage is reduced from 70% to 20%. Measures such as adding reinforcement provide strengthening for both earthquake and hurricane loading.

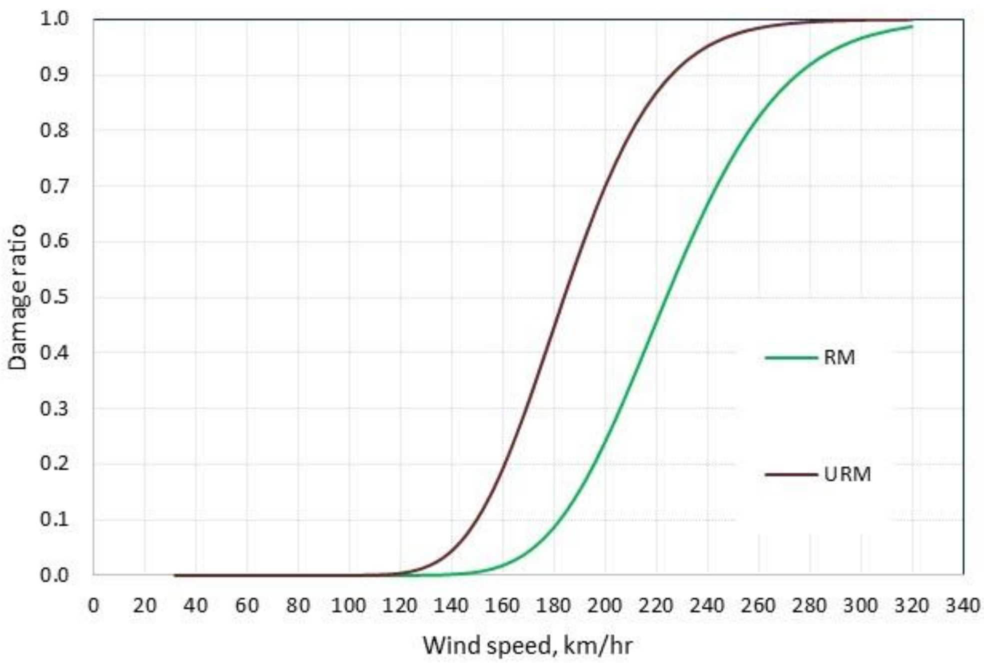


Figure 4-15. Wind vulnerability, water infrastructure (adapted from FEMA, 2003b)

4.6.3 Flood fragility functions

Flood fragility functions for WTP and WWTP are discussed in this section. UGP are not susceptible to damage from flood loading due to their underground location.

Flood hazard consists of inundation, which occurs in cases where the structure's elevation is below the high flood level, scour of foundation due to fast-moving floodwaters, and hydraulic loading. Table 4-15 presents flood vulnerability for water infrastructure, identifying sub-hazards, vulnerable components, and hazard impacts. Figure 4-16 presents damage to WTP and WWTP as a function of flood depth. Note that at a depth of 2 m or greater, over 30% of the infrastructure is damaged. That damage consists of repair of electrical components, cleanup, and repair of buried conduits.

Sub-sector	Flood Hazard Impact			Overall vulnerability	Expected Financial loss	Expected Loss of operation
	Inundation	Scour	Hydraulic loading			
WTP	High	Low	Low	High	High	High
WWTP	High	Low	Low	High	High	High
Underground Pipelines	--	Low	--	Low	Low	Low

Table 4-15. Flood vulnerability, water infrastructure (adapted from FEMA, 2003c)

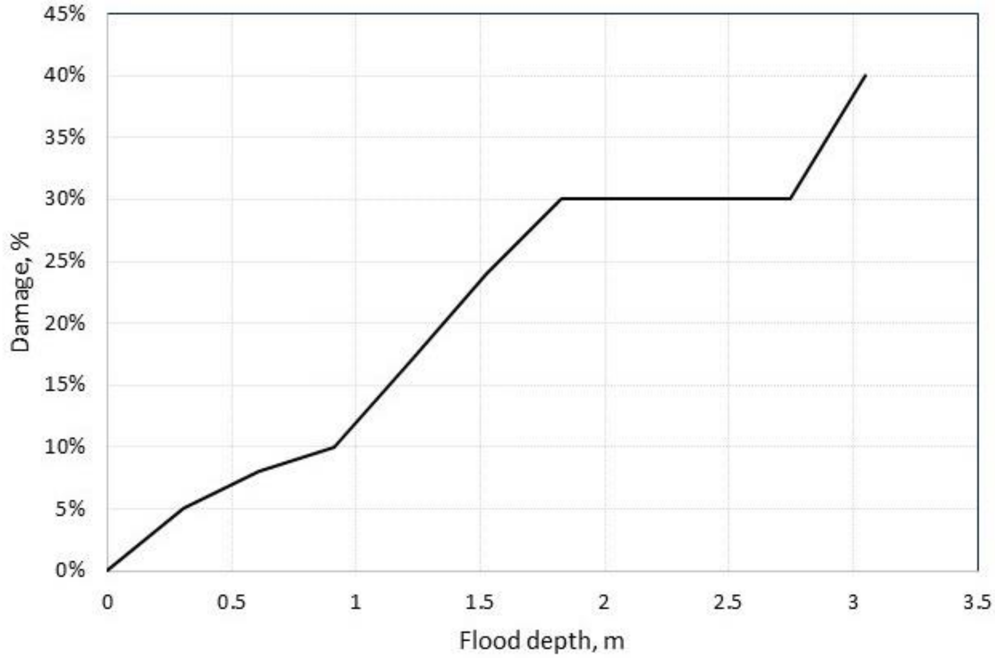


Figure 4-16. Damage to WTP and WWTP as a function of flood depth

4.6.4 Restoration fragility functions

Restoration fragility functions for WTP and WWTP are presented in Figure 4-17. Note that for WTP and WTTP plants:

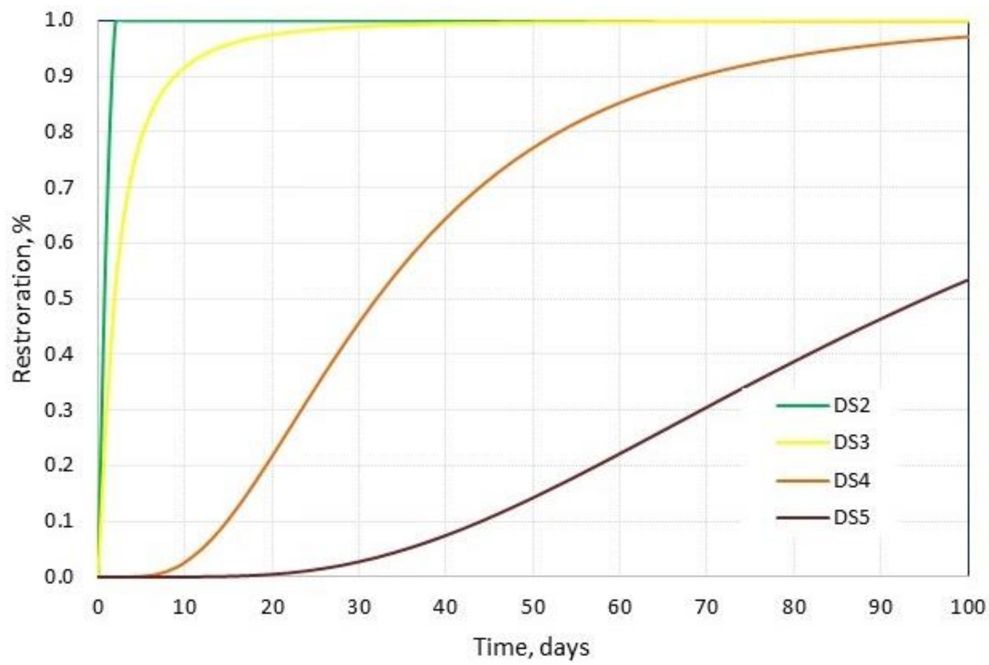
- For WTP plants, the restoration time is reduced from 97% to 47% from extensive to complete damage state.
- For WWTP plants, the restoration time is reduced from 92% to 13% from extensive to complete damage state.

Therefore, by strengthening these facilities, not only are physical damage and associated repair/replacement cost reduced in the aftermath of natural disasters, but there is also a significant decrease in the time to restore services.

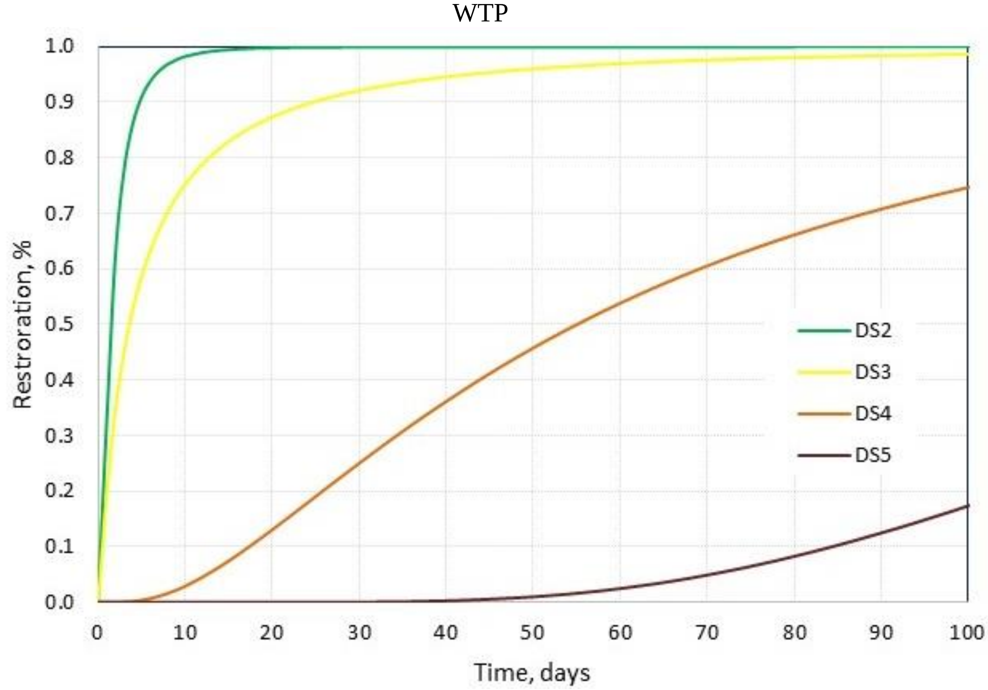
As shown in Figure 4-18, the combined damage from wind and flood exceeds the damage from either natural hazard alone. As such, mitigation measures that address either hazard will also be effective in reducing the combined damage from cases like when hurricanes and ensuing storm surge flooding can damage water and other infrastructure; see Table 4-16. As shown, when proper mitigation techniques are used for multi-natural hazards, the damage ratio is reduced by a factor of four.

Case	Retrofit		Damage, %		
	Hurricane	Flood	Wind	Flood	Total
Existing	No	No	40%	60%	76%
Retrofit 1	Yes	No	10%	60%	64%
Retrofit 2	No	Yes	40%	10%	46%
Retrofit 3	Yes	Yes	10%	10%	19%

Table 4-16. Flood vulnerability, water infrastructure (adapted from FEMA, 2003c)



WTP



WWTP

Figure 4-17. Restoration functions, water infrastructure (adapted from FEMA, 2003a)

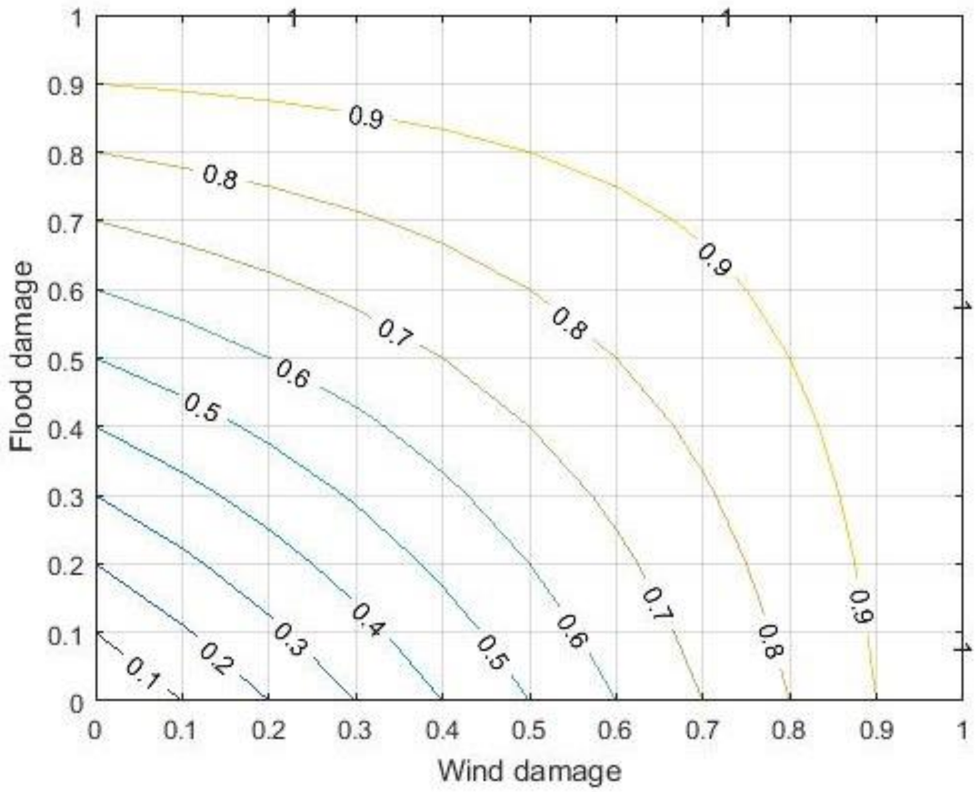


Figure 4-18. Wind and flood combined damage (adapted from FEMA, 2003b)

4.7 Strengthening techniques

4.7.1 Introduction

In the U.S. state of Oregon, a resilience plan for water infrastructure system has been developed. The approach for coastal areas in the state can be applied to the subject countries in this report, given the types of natural hazards. The planned recovery time can be used in the Caribbean to develop strengthening measures to meet these goals.

Examples of proposed recovery goals for water infrastructure are presented in Table 4-17 (OSSPAC, 2013). The Oregon Seismic Safety Policy Advisory Commission (OSSPAC) notes that:

> *If it were to occur today, a Cascadia subduction zone earthquake would result in catastrophic impacts to existing water and wastewater systems throughout western Oregon.*

Case	**Recovery time**				
	1-3 days	**1 week**	**2 weeks to 1 month**	**1-3 months**	**3-6 months**
Water to WTP		20-40%	50-60%		80-90%
Main pipes	20-40%	50-60%	80-90%		
WWTP operational			20-40%	50-60%	80-90%

Table 4-17. Recovery times (OSSPAC, 2013)

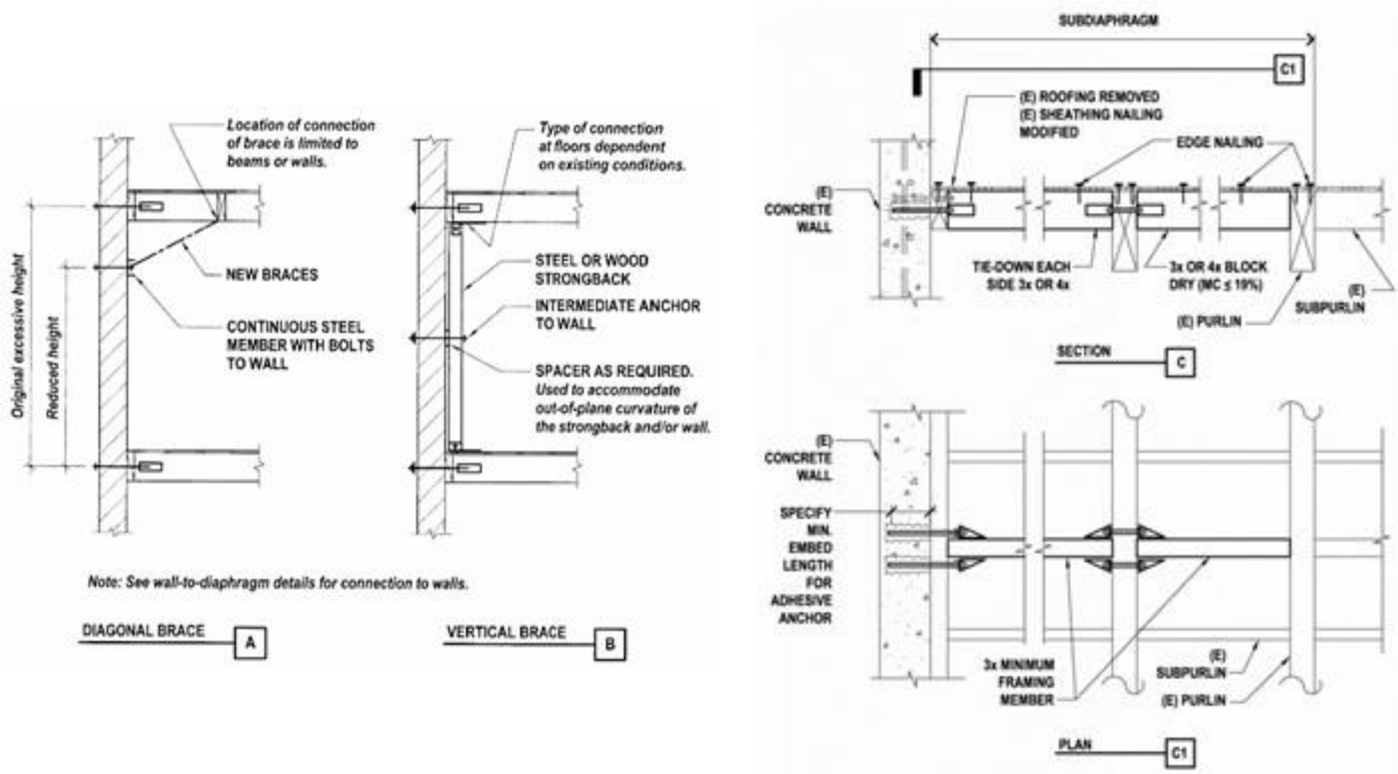
4.7.2 WTP and WWTP

Plant buildings, water storage, and equipment (mechanical and electrical) are identified as representative facilities of WTP and WWTP infrastructure. According to the type of hazard, the typical strengthening measures mainly implemented in the U.S. are discussed in this section for the vulnerable facility components listed in Table 4-8.

4.7.2.1 Earthquake: ground shaking

4.7.2.1.1 Wall out-of-plane strengthening (Plant building)

Old or unreinforced buildings generally do not possess appropriate steel reinforcement in masonry/concrete walls to resist seismic out-of-plane force. Thus, the out-of-plane bending resistance of the walls is usually inadequate. The failure of heavy masonry/concrete walls due to a lack of out-of-plane strength is the most devastating damage to buildings. Bracing the wall at properly-spaced intervals is a general measure for improving the resistance of wall out-of-plane failure. To stiffen the wall, a steel strongback or brace supporting at diaphragm (i.e., horizontal floor element) and post-installed anchor are typically applied; see Figure 4-19. This improvement can be installed on both the inside and outside of the building, but the inside approach is more popular for property-line limitation or architectural exterior finish restrictions.

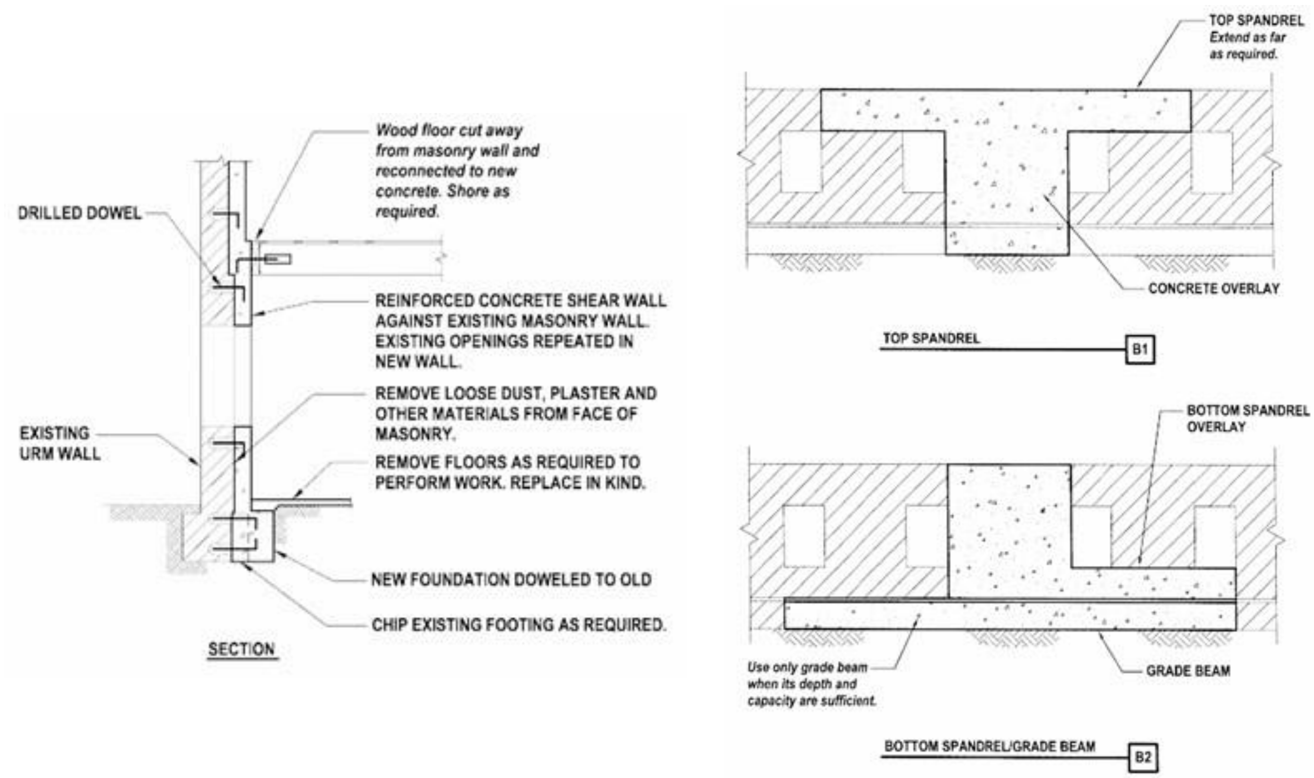


Out-of-plane wall bracing — Anchorage from diaphragm to wall

Figure 4-19. Wall out-of-plane strengthening (FEMA, 2006b)

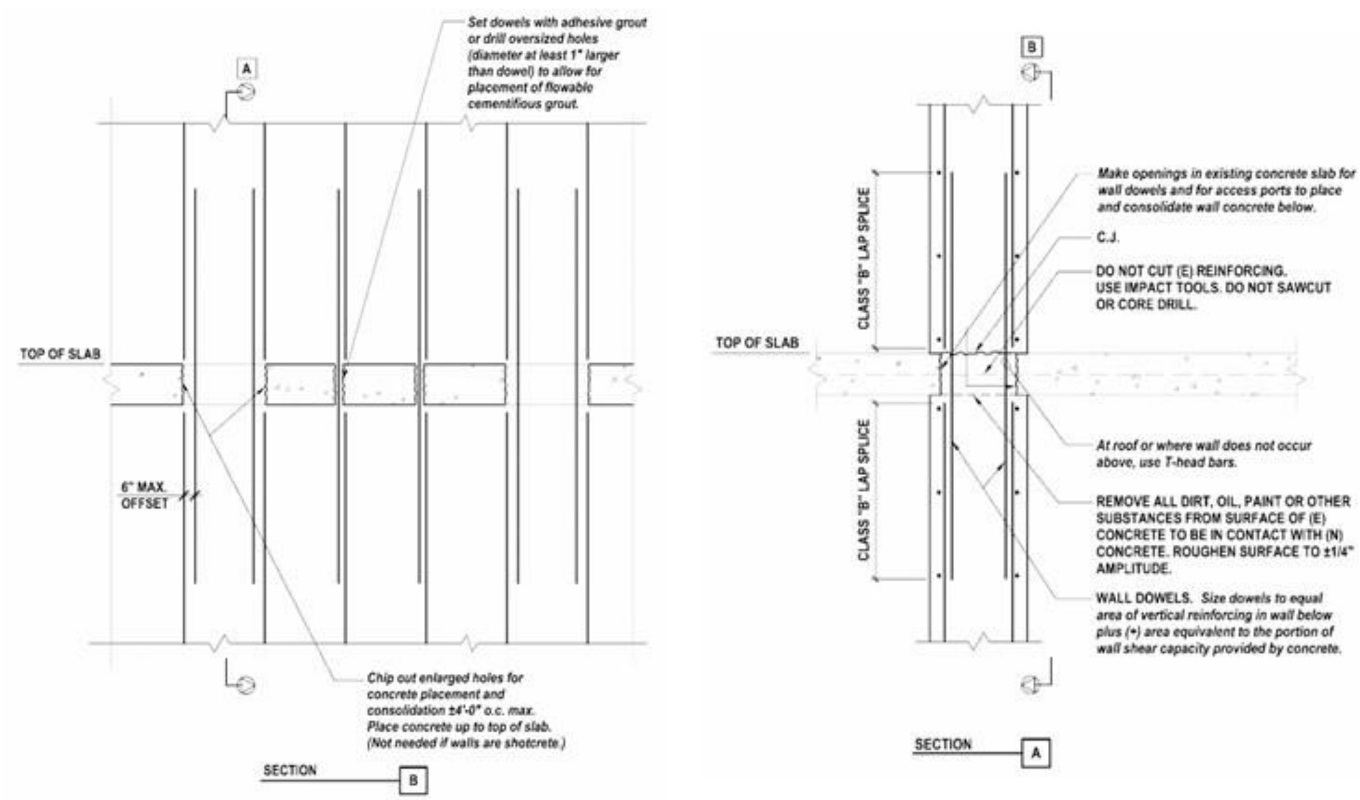
4.7.2.1.2 In-plane strengthening (Plant building)

The global seismic strength of a building heavily relies on the in-plane strength of vertical elements, such as shear walls, braced frames or moment frames. To improve the in-plane seismic strength of a masonry/concrete building, a new element, such as a concrete shear wall, is typically added to the existing structural system. Adding new elements from the outside is more convenient for plant functionality, building usability and retrofit constructability, but inside installation could be constructible with a careful installation plan. Typical configurations and details of reinforced concrete (RC) shear walls overlaying on existing masonry wall and adding to existing building are shown in Figure 4-20 and Figure 4-21, respectively. In both cases, it is important to provide the appropriate load path elements to transfer the seismic force from diaphragm to the ground through the new wall elements and foundations.



Concrete/shotcrete wall overlay — Wall addition elevation

Figure 4-20. RC shear wall overlay on existing masonry wall (FEMA, 2006b)

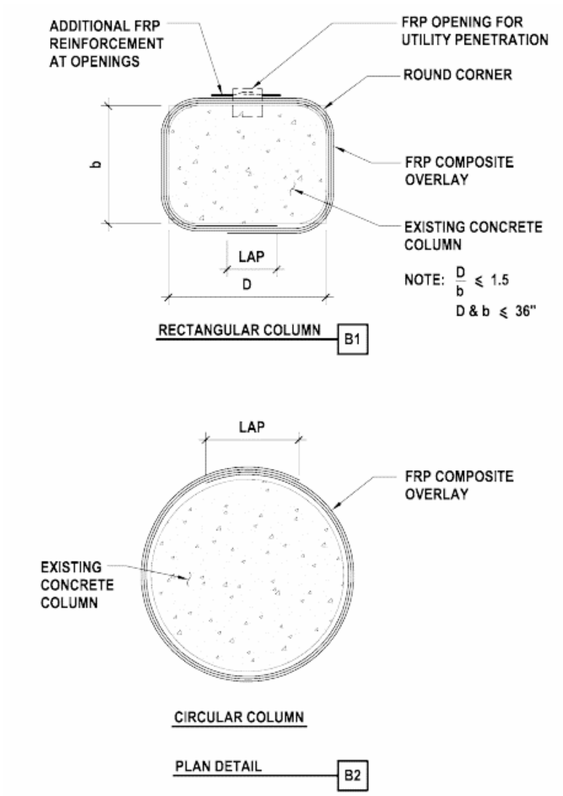


Wall elevation with existing slab — Wall connection with existing slab

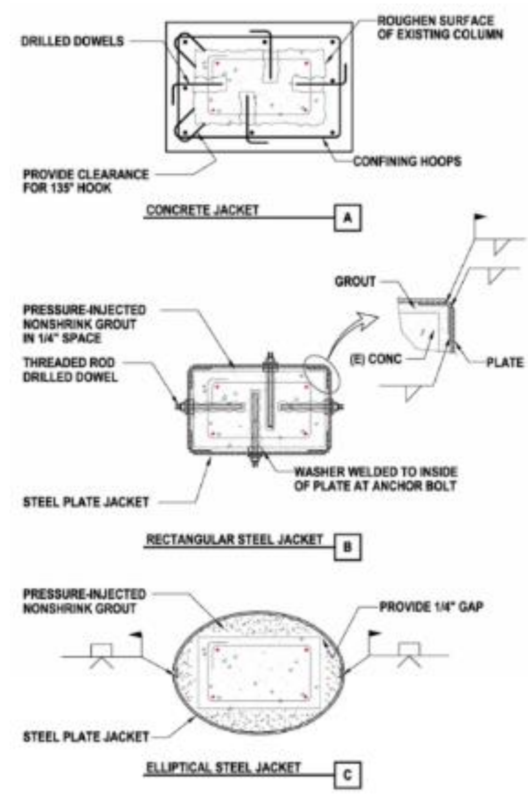
Figure 4-21. RC shear wall addition to existing building (FEMA, 2006b)

4.7.2.1.3 Ductility improvement (Plant building)

Ductility of a building during an earthquake is simply described as the capability of large rotation and displacement without any serious damages. Insufficient ductility is typically caused by an inadequate confinement of structural elements (e.g., column, beam or wall) or an inappropriate yielding location at the column-beam connection. To ensure the ductile behavior of masonry/concrete buildings during earthquakes, some retrofits should be implemented on local elements. For concrete columns or masonry piers, for example, FRP overlay, concrete jacketing or steel jacketing are popular measures to improve the confinement of existing columns, to increase the capacities for shear and axial loads and to enhance ductility, as presented in Figure 4-22.



Column retrofit by FRP overlay

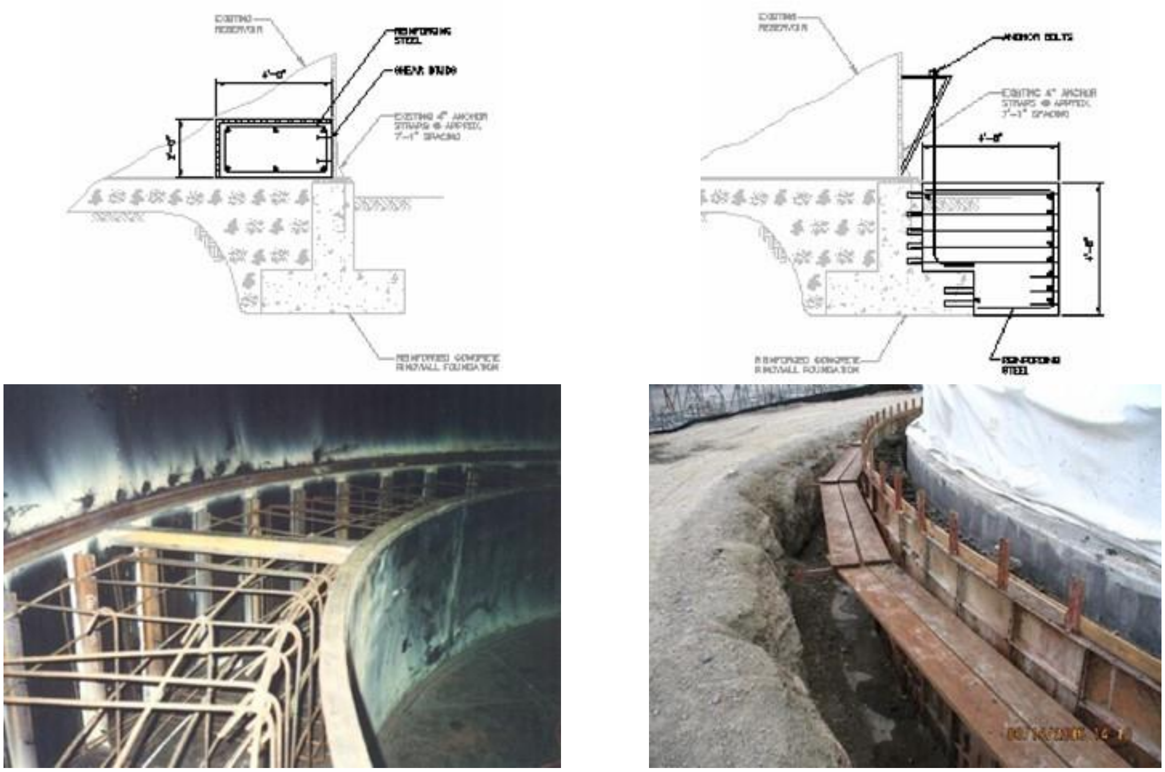


Column retrofit by concrete/steel jacketing

Figure 4-22. Ductility improvement for columns (FEMA, 2006b)

4.7.2.1.4 Foundation strengthening (Water storage)

Seismic damage to water storage tanks is typically induced by unstable uplift and compression movements caused by the inertial force of the water tank due to earthquake shaking. This movement could induce tank wall buckling (i.e., elephant foot damage), foundation uplift and failure of the steel strap between the tank and foundation. To prevent these damages and stabilize the water tank during an earthquake, increasing weight resistance (i.e., mass addition) is a common way for withstanding potential uplift and compression movements, as presented in Figure 4-23. In the left drawing and picture, an internal ballast ring retrofit is shown. A proper connection (e.g., welded stud connector) between the concrete ballast ring and the tank wall is needed and the tank must be out-of-service during retrofit construction. In the right figures, an external concrete foundation addition is presented. Sufficient anchorage between the tank and new foundation is necessary, and the tank may remain in service, as this retrofit can be externally constructed.

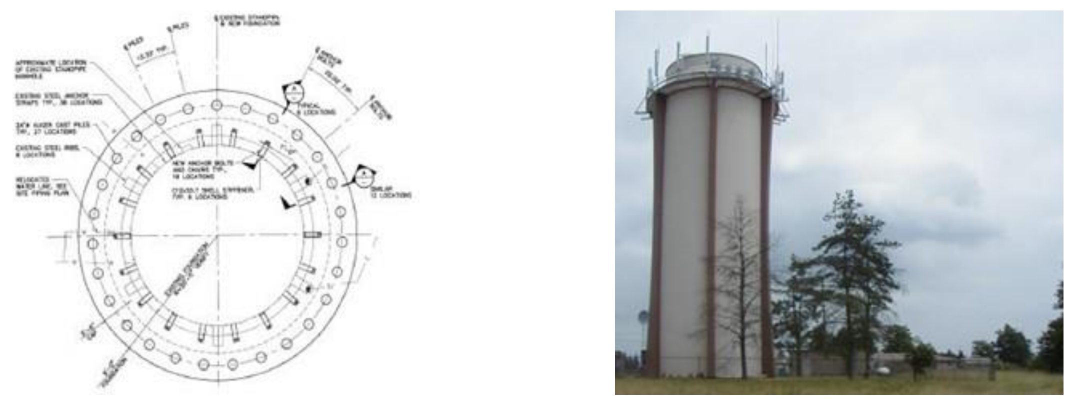


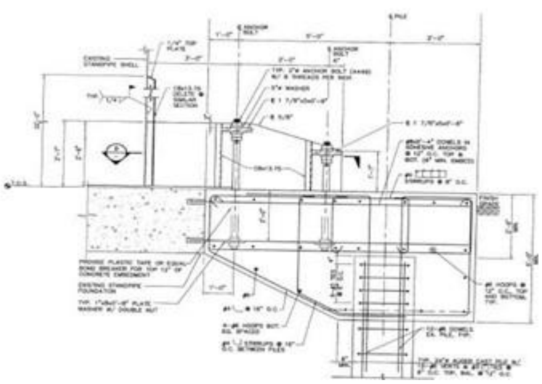
Internal ballast ring External anchor ring

Figure 4-23. Foundation strengthening of water storage tank (Porter, 2008)

4.7.2.1.5 Anchorage improvement (Water storage)

As described in the previous section, the connection between the water tank and foundation is also important to mitigate potential seismic damage of the water tank. As exemplified in Figure 4-24, a water tank with a high-aspect ratio usually needs a large amount of anchorage strength and foundation uplift capacity, because the extensive uplift force due to overturning moments can be expected during an earthquake. Large-sized anchor bolts are installed in the additional deep foundation in this example. The necessary capacity of anchorage elements and the foundation to resist a design seismic uplift should be identified by considering the local seismicity, soil characteristics and water tank volume for the design.





Retrofit drawings

Tank and added anchors

Figure 4-24. Anchorage improvement with pile addition on water storage tank (Porter, 2008)

4.7.2.1.6 Foundation-tie retrofit (Equipment)

WTP/WWTP facilities usually contain mechanical and electrical equipment, such as water pumps, power generators, filtering systems and so forth. This equipment is susceptible to horizontal shear force and overturning moments due to earthquake ground shaking, and those forces are sometimes largely amplified depending on the installation height of equipment. Several design guidelines have been published for the seismic design of equipment in countries like the U.S. or Japan. Based on the guidelines, representative retrofits aim to improve the connection between equipment and base foundation in order to resist seismic shear force and overturning. Adding rigid anchorage or installing vibration isolators at the equipment base are shown in Figure 4-25 and Figure 4-26, respectively. These retrofits improve the robustness and stability of equipment support against earthquake shaking.

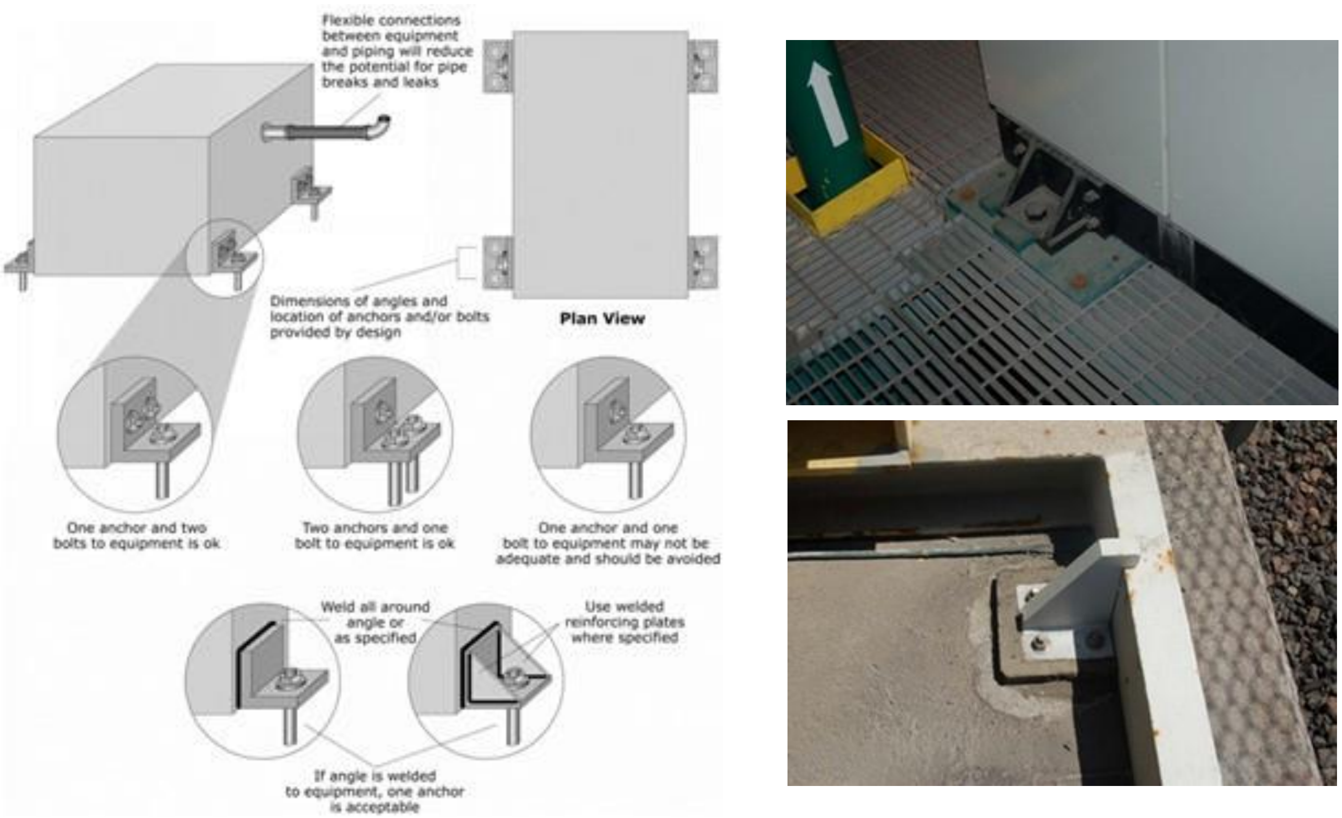


Figure 4-25. Rigid anchorage strengthening on equipment (FEMA, 2011)

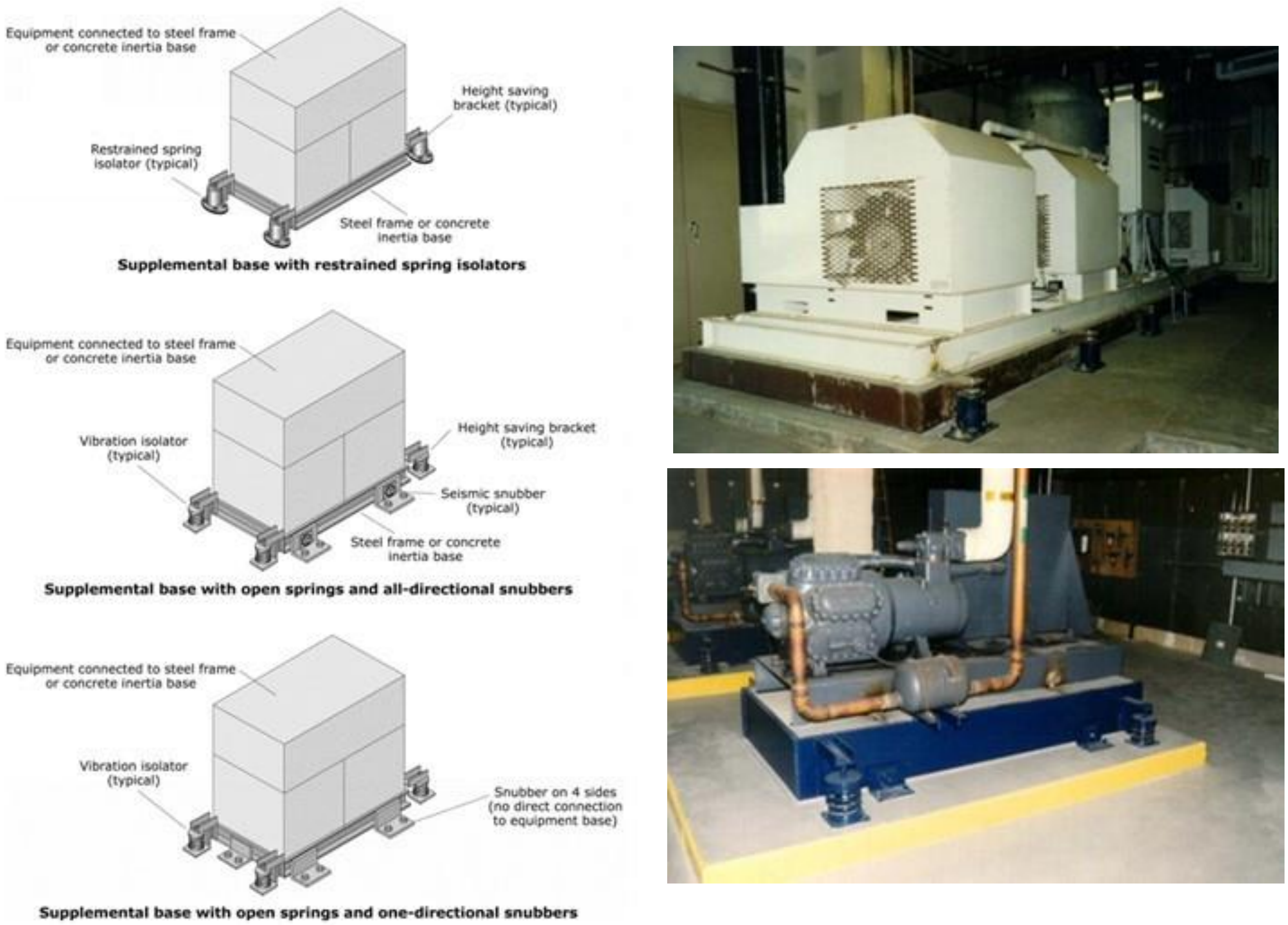
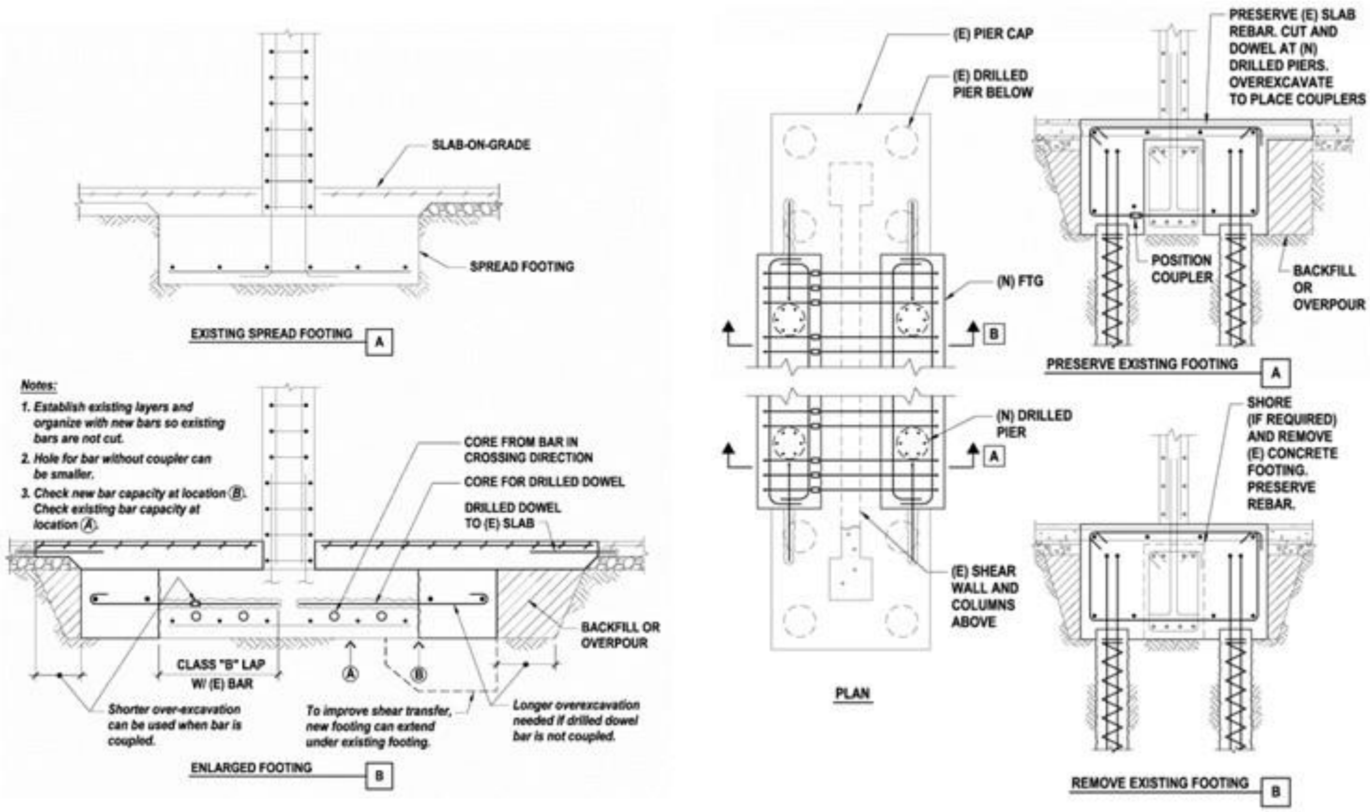


Figure 4-26. Vibration isolator support on equipment (FEMA, 2011)

4.7.2.2 Earthquake: liquefaction

4.7.2.2.1 Foundation retrofit

When the size of shallow-spread foundations or the amount of deep-pile foundations are not sufficient to support the building, structures, facilities or equipment for a liquefaction event during an earthquake, the consequence could be demolishment and replacement, or extensive repair work with foundation improvement accompanying a long-term functional outage. As shown in Figure 4-27, enlarging the existing footing by attaching additional concrete footing is a typical measure to increase capacity for shallow-spread footing, and new pile installation next to an existing deep-pile foundation is usually implemented to improve deep-foundation capacity. In both cases, the retrofit construction work is not simple because, for example, excavation, formwork, reinforcement placement and concrete pouring are usually required around the existing foundation.

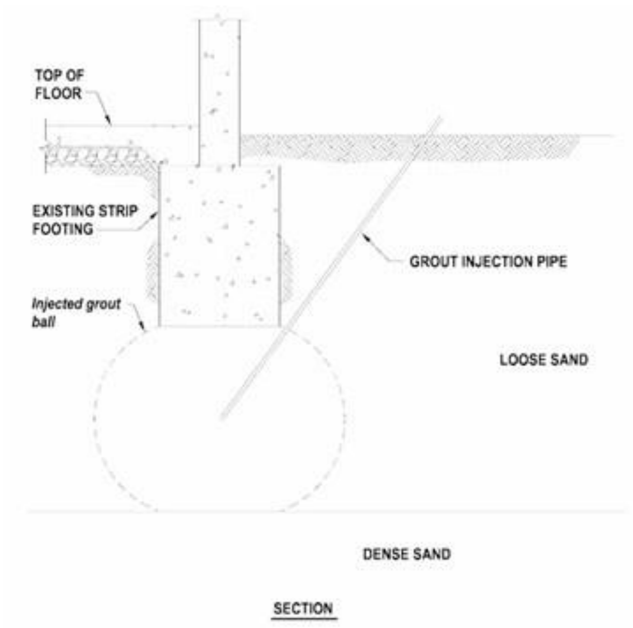


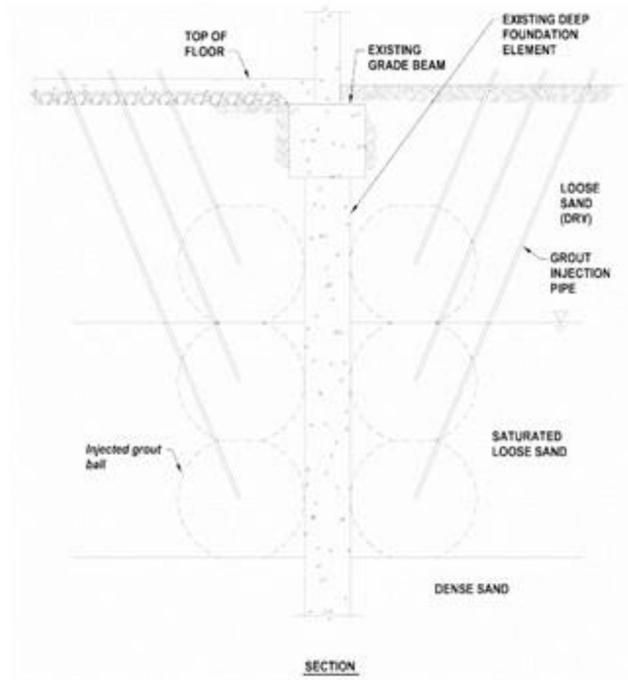
Enlarge existing spread footing Adding piles to existing deep foundation

Figure 4-27. Typical foundation strengthening measures (FEMA, 2006b)

4.7.2.2.2 Soil improvement

Soil improvement is another approach to keep buildings, structures, facilities or equipment stable from large ground failure due to liquefaction. In Figure 4-28, a measure to improve the soil below existing shallow foundations and around existing deep foundations uses permeation grouting on liquefiable soil layers. There are generally two types of soil grouting: permeation grouting and compaction grouting. The former injects chemical/cement grout into the soil and aggregate without displacing the materials, and it solidifies the sandy soils and improves resistance to liquefaction. The latter injects very stiff grout into a layer of soil to force the soil particles into a tighter packing arrangement, and it increases the soil density and the liquefaction resistance (FEMA, 2006b).





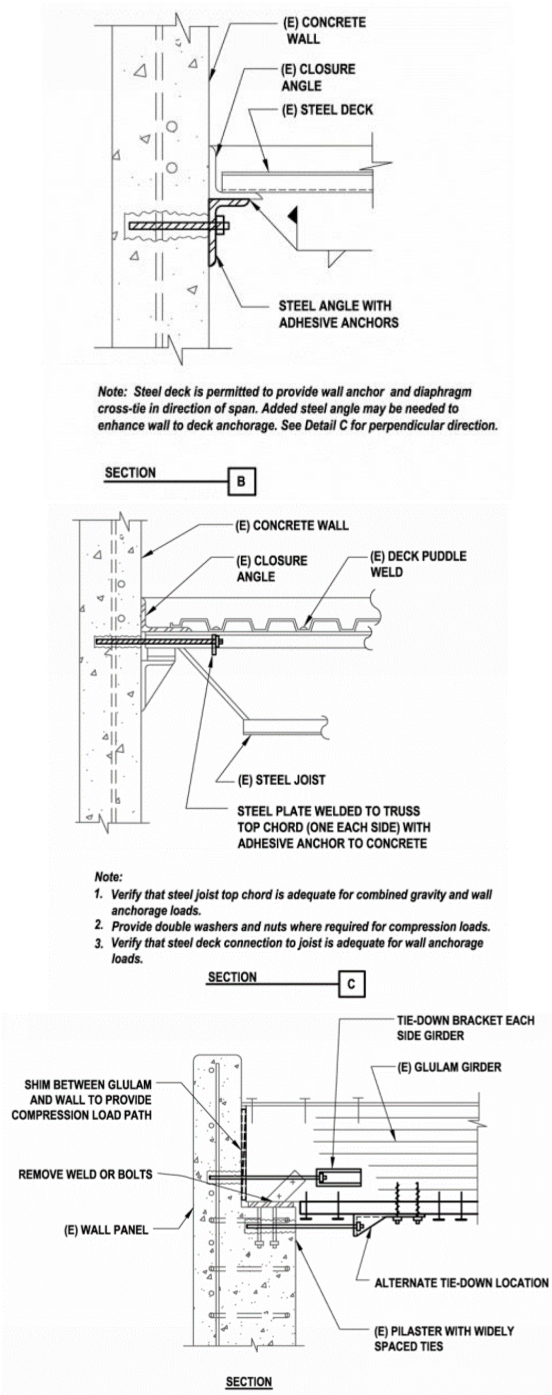
Under existing shallow foundation   Around existing deep foundation

Figure 4-28. Soil improvement on liquefiable soil layers around existing foundation (FEMA, 2006b)

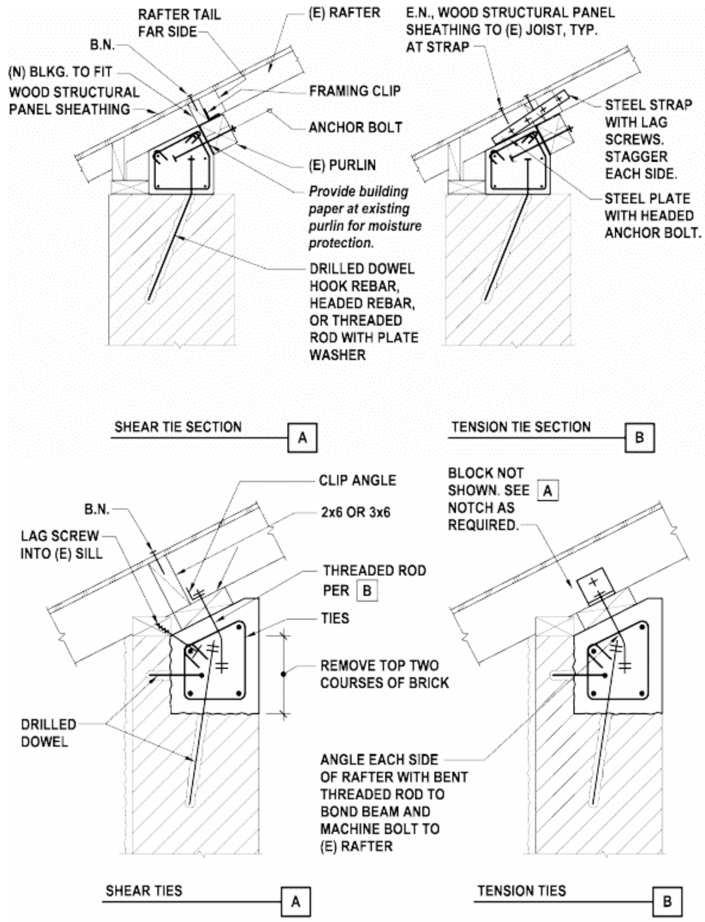
4.7.2.3 Wind

4.7.2.3.1 Roof connection strengthening (Plant building)

Strong wind can easily blow the roof away if it is not securely attached to the walls or beams of a building. Since the roof usually has a structural role in horizontally supporting a wall, losing the roof due to strong wind might cause cascading damage, like wall instability or failure. Typical details to strengthen the roof connection of concrete and masonry structures are shown in Figure 4-29. Adding steel angles and anchors to strengthen the roof connection is a general measure for concrete structures. For masonry structures, new concrete bond beams might be needed to enhance a connection between the masonry wall and roof, and anchoring, steel strap and nailing are typically provided for securing the roof connection.



Roof connection of concrete structure



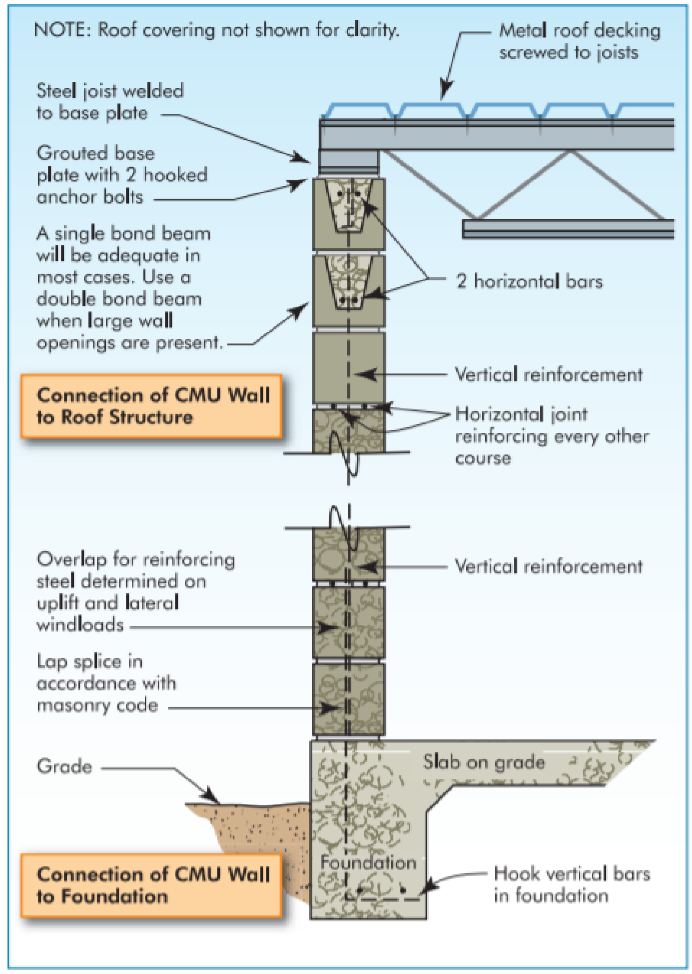
Roof connection of masonry structure

Figure 4-29. Typical roof connection improvements (FEMA, 2006b)

4.7.2.3.2 External wall improvement (Plant building)

When an external wall is not firmly attached to a building, fall or collapse of external non-structural walls is a typical failure mode in a strong wind event. If it is not a structural element or bearing wall, it could just cause minor damage, structurally, to a building, but might also induce severe functional damage to the plant building. In Figure 4-30, hurricane damage to perimeter block walls and a typical connection detail of block wall, roof and foundation are presented. An adequate connection with anchor and reinforcement shown in the figure on the right could avoid this type of wall failure.

Hurricane damage to external wall



Typical connections of external wall

Figure 4-30. Connections of external wall (FEMA, 2007a)

4.7.2.3.3 Foundation-tie retrofit (Equipment)

As well as the foundation-tie retrofit for equipment against earthquake shaking (see Section 4.7.2.1.6), mechanical and electrical equipment are also susceptible to shear force and overturning moments due to strong wind. Equipment with larger areas, taller heights and/or light weight (which likely causes severe uplift) is more sensitive to wind forces. An effective retrofit would strengthen the base anchorage of equipment, as presented in Figure 4-31. This type of rigid anchorage for resisting uplift force may require additional foundational uplift capacity; foundation improvement is then also needed in that case.

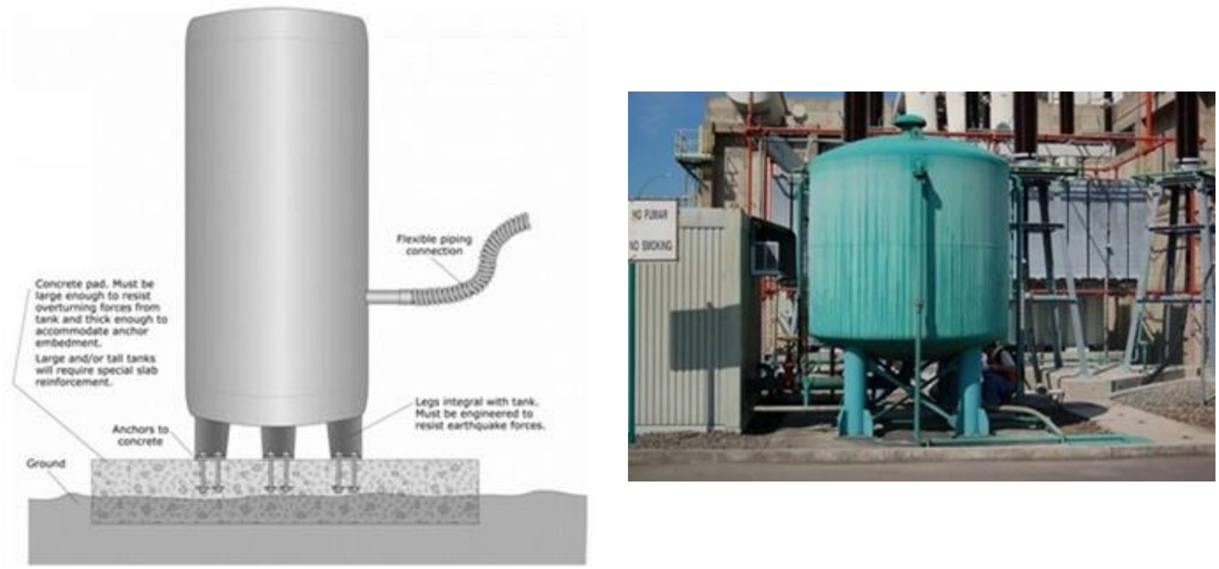


Figure 4-31. Anchorage strengthening of equipment (FEMA, 2011)

4.7.2.4 Flood

4.7.2.4.1 Building envelope improvement (Plant building)

One of the retrofit measures to keep a building protected from flood inundation is installation of floodgates at potential water intrusion routes around the building and flood shields at openings of the building, such as doors and windows; see Figure 3-53. The height of the gate or shield should be designed according to the expected flood depth analyzed by local flood hazard conditions, and the gate and shield need to be structurally designed to resist the water pressure due to inundation depth. Preventing water inundation could help avoid any functional outages of the plant building that are potentially caused by damage to mechanical and electrical equipment inside of the building.

4.7.2.4.2 Elevated construction installation (Plant building)

Constructing an elevated structure, like a floodwall around a building or facility site, is also an effective measure to protect a building from flood inundation. Typical types of floodwalls are shown in Figure 3-55. A floodwall is typically constructed some distance from a building, and it should be designed as a freestanding, independent structure separated from the building. This kind of wall is generally built of reinforced concrete. For designing a floodwall, the lowest point of the site, topographic information, the site-specific flood hazard level (i.e., inundation depth), soil characteristics, potential debris impact and a performance objective need to be identified. A successful example of a floodwall is presented in Figure 4-32; the floodwall provides a perfect barrier for the building and site against flood inundation.

Figure 4-32. Floodwalls protecting buildings and facility site (FEMA, 2013)

4.7.2.4.3 Elevating equipment (Equipment)

To avoid functional loss of mechanical or electrical equipment at WTP/WWTP caused by floodwater, locating the equipment above the regionally-expected flood height is one common countermeasure. Unlike a building or large structure, elevating mechanical or electrical equipment is not very difficult, as the equipment can be basically moved, and relocating equipment is practicable. Based on the regional design flood height and the elevation of the WTP/WWTP site, the lowest height of equipment should be decided, and then the equipment relocated onto elevated steel platforms or put on an upper floor, as shown in Figure 4-33.

Elevated equipment on upper floor (FEMA, 2013)

Elevated equipment platform (Hilderhoff, 2013)

Figure 4-33. Elevating equipment for flood protection

4.7.2.4.4 Dry floodproofing (Equipment)

Similar to floodwalls or flood shields on buildings, watertight shields for mechanical/electrical equipment are a typical dry floodproofing measure. As presented in Figure 4-34, the watertight shield can be applied to both an inside equipment room and an outside equipment station. These watertight doors and barriers need to be designed by the static and dynamic water pressures evaluated on the design flood level. Also, in this case, a route to access the equipment during flooding has to be secured for checking or maintenance purposes, as this is not a dry floodproofing measure for the entire WTP/WWTP site, but for the limited area of equipment.

Watertight door (FEMA, 2013)

Watertight barrier (Hilderhoff, 2013)

Figure 4-34. Watertight shields for equipment for dry floodproofing

4.7.3 Underground pipes

For underground pipes transmitting potable water in the subject countries, there are a number of strengthening methods available. These techniques are discussed in this section for various natural hazards.

4.7.3.1 Earthquake

4.7.3.1.1 Overview

Underground pipelines in the Caribbean are subject to damage from both PGV and PGD; see section 4.6.1.4. Replacement of brittle pipes (see Figure 4-35) with ductile pipes (see Figure 4-36) is the most economical and effective option in the Caribbean. Experience has shown that most failures (breaks and leaks) occur in brittle pipes. Additionally, for asbestos-cement (AC) pipes, the pipe wall thickness decreases when in use and also loses structural integrity due to leaching, a process where the calcium content of the pipe is absorbed by the surrounding soil. Cast-iron (CI) pipes are also subject to corrosion and leaks. Concrete pipes can experience damage at the joints.

Asbestos cement

Cast iron

Concrete

Figure 4-35. Examples of brittle pipes (various)

High-Density Polyethylene (HDPE)

Ductile iron (DI)

Welded steel (WS)

Figure 4-36. Examples of ductile pipes (various)

Eidinger and Davis (2012) documented damage to various types of pipelines in recent earthquakes. Table 4-18 summarizes their findings.

Damage description	**Chile 2010**	**New Zealand 2011**	**Japan 2011**
Damage to large-diameter transmission pipes	Many	N/A	Many
Damage to small-diameter distribution pipes (AC, CI)	Thousands	Thousands	Thousands
Damage to HDPE distribution pipe	None	None	None
Damage to DI with Earthquake-resistant joints	N/A	N/A	None

Table 4-18. Damage to various pipelines in recent earthquakes (adapted from Eidinger and Davis, 2012)

Eidinger and Davis (2012) write:

> *Chile 2010: Strong ground shaking and liquefaction damaged the city's only water treatment plant. Liquefaction damaged many of the larger diameter steel transmission pipelines. Liquefaction damaged many distribution water pipelines (about 3,000 total). Water outages to customers were lengthy, over a month to some customers. Even so, there was some good news: Essbio (the water system operator) had been installing pipe in its water distribution system for about a decade prior to the earthquake; while the rest of the water system suffered thousands of damaged pipes, no HDPE pipe was damaged.*

New Zealand 2011: Strong ground shaking and liquefaction damaged many of the city's wells. Liquefaction damaged many distribution water pipelines. The city's largest concrete reservoir failed completely due to ground deformations. Water outage durations to customers were moderate, mostly restored within 10 days after each earthquake. Even so, there were some good lessons learned: The Christchurch City Council (the water system operator) had installed some HDPE pipe in its water distribution system after the first earthquake; in the subsequent earthquakes, no HDPE pipe was damaged, while nearby older pipes were damaged.

Japan 2011: A large diameter water transmission pipeline in the epicentral area suffered major damage at more than 50 locations, mostly due to pulled slip joints. A few large diameter water transmission pipelines suffered some slip joint damage in low-shaken areas, very distant from the earthquake. None of the seismic-resistant pipelines is known to have been damaged.

4.7.3.1.2 Risk analysis

Table 4-19 presents the proposed seismic retrofit solutions for the underground pipes in the subject countries. Note that based on the level of seismicity, the soil condition, and the importance of pipe, a number of strengthening measures are recommended.

Pipe importance	Soil profile	PGA	Strengthening measure
Critical (high capacity, serving large population) or pipes with 3 or more leaks in last 5 years	Poor	≥0.3 g	Replace with HDPE, WS, DI
		<0.3 g	Upgrade joints
	Competent	≥0.3 g	Upgrade joints
		<0.3 g	Monitor
Other	Poor	≥0.3 g	Upgrade joints
		<0.3 g	Monitor
	Competent	≥0.3 g	Upgrade joints
		<0.3 g	Monitor

Table 4-19. Suggested earthquake strengthening measures for UGP in the subject countries

4.7.3.1.3 Pipe replacement

Existing brittle pipes can be replaced with ductile pipes. This retrofit can be implemented when a pipe segment is due for replacement.

San Mateo County in California conducted a water system seismic improvement feasibility study (WWE, 2015). They noted that the existing water distribution system was vulnerable and needed upgrades. The proposed retrofits included:

The majority of the existing La Honda water distribution system was constructed in the 1920s' using AC and galvanized steel (GS) pipe materials In past, regular pipe breakage and leakage occurred throughout the system. The repairs were made using temporary fittings and clamps to keep the system in operation with a minimum downtime. WWE identified deficiencies in the existing water system, quantified necessary capital improvements, and analyzed various alternatives in the next section.

Pipelines: replace with HDPE or ductile iron pipe

The American Lifelines Alliance (ALA) provides guidelines (2005) for seismic design of pipelines. The proposed criteria can be used when considering replacement of existing pipes in the Caribbean for the purpose of improved seismic performance. The document writes:

These Guidelines are not intended to completely eliminate all seismic induced pipe damage for Function Class II, III and IV pipelines, but it will significantly reduce the damage and *post-earthquake recovery time. In addition, the ability for the system to perform during and following an earthquake will be significantly improved.*

ALA (2005) emphasizes the importance of redundancy, defined as the following:

1. A leak or break in one pipe will not likely lead to damage on other redundant pipes;

2. All redundant pipes can provide a minimum needed flow to meet post-earthquake operational needs. The minimum level of flow required after earthquakes should generally be at the maximum winter time flow rate, or a level of water that is sufficient for household and most economic activities of the community

3. The redundant pipes are spatially separated by an adequate distance through potential ground deformation zones (landslide, fault movement, ground failure, lateral spreading, etc.) such that, should ground deformation occur, each redundant pipe may not be subjected to the same amount of ground movement due to the natural variation in movement

The pipes are classified into four (4) categories and for each category, specific design recommendations are provided; see Table 4-20.

Class	Description	Redundancy	Earthquake	Liquefaction consideration	PGD for 500-year earthquake
I	Pipelines that represent very low hazard to human life in the event of failure. Not needed for post-earthquake system performance, response, or recovery.	0	--	--	--
I		1	--	--	--
I		2	--	--	--
II	Normal and ordinary pipeline use, common pipelines in most water systems.	0	500 year	High, Very High	1.0*PGD
II		1	500 year	High, Very High	1.0*PGD
II		2	500 year	High, Very High	1.0*PGD
III	Critical pipelines serving large numbers of customers and present significant economic impact to the community or a substantial hazard to human life and property in the event of failure.	0	1000 year	High, Very High	1.35*PGD
II		1	500 year	High, Very High	1.0*PGD
II		2	500 year	High, Very High	1.0*PGD
IV	Essential pipelines required for post-earthquake response and recovery and intended to remain functional and operational during and following a design earthquake.	0	2500 year	Moderate, High, Very High	1.0*PGD
III		1	1000 year	High, Very High	1.35*PGD
II		2	500 year	High, Very High	1.50*PGD

Table 4-20. Recommended seismic design for different classes of pipelines

4.7.3.1.4 Joint retrofit

The stiff joints in critical pipe sections can be replaced with flexible joints that allow for movement of one section of pipe relative to the adjacent segment. San Mateo County in California conducted a water system seismic improvement feasibility study (WWE, 2015). They noted that some of the 100-mm AC pipe connections had unrestrained and inflexible connections. The proposed retrofits included:

Incorporates an EBAA Iron force‐balanced Flex Tend coupling that has an ability to deflect up to 20 degrees in all directions and can move laterally or horizontally for up to 200 mm. sufficient room for the force balanced Flex Tend coupling movement shall be provided. Use a Romac Alpha coupling that has an ability to deflect up to 8 degrees in all direction but the lateral and horizontal movement is limited to only 8 degrees of deflection.

The International Organization for Standardization (ISO) provides the seismic performance requirements (2006) for ductile joints; see Table 4-21. Note that for seismic applications, connections with high expansion/contraction capacity, slip-out resistance, and joint deflection should be specified.

Parameter	Class	Performance
Expansion contraction	S-1	≥1% length
	S-2	<1% length
	S-3	<0.5% length
Slip-out resistance	A	≥3*diameter kN
	B	≥1.5*diameter kN
	C	≥0.75*diameter kN
	D	<1.5*diameter kN
Joint deflection angle	M-1	≥15 degrees
	M-2	≥7.5 degrees
	M-3	<7.5 degrees

Table 4-21. Suggested earthquake strengthening measures for UGP in the subject countries

4.7.3.2 Liquefaction

The strengthening techniques for liquefaction ground deformation are the same as the options for earthquake mitigation. Table 4-22 presents the proposed seismic retrofit solutions for underground pipelines in the subject countries. Note that based on the level of seismicity, the soil condition, and the importance of pipe, a number of strengthening measures are recommended.

Pipe importance	Soil profile	PGD	Strengthening measure
Critical (high capacity, serving large population) or pipes with 3 or more leaks in last 5 years	liquefiable	≥50 mm	Upgrade joints
		<50 mm	Monitor
	Competent	≥50 mm	Monitor
		<50 mm	--
Other	liquefiable	≥50 mm	Monitor
		<50 mm	--
	Competent	≥50 mm	--
		<50 mm	--

Table 4-22. Suggested liquefaction strengthening measures for UGP in the subject countries

4.7.3.3 Wind

Water pipelines are not adversely affected by windstorms due to their location.

4.7.3.4 Flood

Water pipelines can experience large displacements due to buoyancy effects only if the pipelines are empty (for example, during maintenance or repair), but it is quite rare. When failures of this kind occur, the discussion in the liquefaction-related section should be followed.

4.8 Cost analysis

4.8.1 Introduction

Water and wastewater utilities face competing economic constraints. To implement strengthening techniques, it is critical to undertake a comprehensive plan that includes the following components:

- Perform a screening of water infrastructure and detail their vulnerabilities
- Prioritize the most vulnerable water infrastructure components and schedule them for upgrade
- Develop a cost estimate for strengthening
- Perform a feasibility study on the benefits of the strengthening of water infrastructure

It is noted that some cost estimates presented in this section are for the United States. However, it is anticipated that the ratio of strengthening-to-reconstruction costs are similar in the U.S. and the Caribbean.

4.8.2 Reconstruction costs

The Caribbean Environmental Health Institute (CEHI, 2013) conducted a study regarding the financial assessment of wastewater treatment in the Caribbean. They noted that:

- There have been no wastewater management assessments for a decade;
- There is little financial cost data for developing appropriate systems;
- There has been more focus on water management;
- Despite this, there are technologies for wastewater systems in the Caribbean.

They developed cost estimates based on assumptions that urban and rural areas were included, that there was no limitations on space availability, and based on assumptions of per capita wastewater rates. Table 4-23 presents cost assumptions used for small systems. Table 4-24 presents the funding requirements based on the above assumptions.

Item	Cost, US$
Waste Stabilization Ponds	1,100 per m^3
Equipment and installation	7,800 per m^3
Pipe network	650 per m^3
Lift stations	160 per m^3

Table 4-23. New wastewater infrastructure cost data (adapted from CEHI, 2013)

Region	Continental	Large island	Small island
Countries	Belize, Guyana, Suriname	Trinidad and Tobago, Bahamas, Haiti, Jamaica	Antigua, Barbados, Dominica, Grenada, Saint Kitts and Nevis, St. Vincent and the Grenadines, St. Lucia
Representative country	**Guyana**	**Jamaica**	**Saint Lucia**
People served	600,000	465,000	160,000
Daily flow, m^3	750,000	106,000	37,000
Construction, US$M per m^3	4,300	2,100	37,000
Operation and maintenance, US$M per m^3	140/year	270/year	10000/day

Table 4-24. Cost estimates for example pilot countrywide projects, wastewater (adapted from CEHI, 2013)

The World Health Organization (WHO, 2008) examined the cost of improving water infrastructure to reduce the number of people without access to potable water and wastewater sanitation by half. The data was prepared for various regions. The relevant data for the Americas are presented in Table 4-25. This data and the population in the subject countries can be used to estimate the new construction cost for water infrastructure in the Caribbean.

Water improvement		Sanitation improvement	
Initial cost	**Annual recurrent cost**	**Initial cost**	**Annual recurrent cost**
72	0.5	134	5.0

Table 4-25. Per capita cost in US$ of water and wastewater improvement and access (adapted from WHO, 2008)

The Caribbean Development Bank (CDB, 2020) has approved a loan of US$6,100,000 for improvement of water network projects in Dominica. Table 4-26 presents the components covered as part of this project.

Component	Description
Water storage tank	100,000 gallon reinforced concrete
	250,000 steel
Pipeline	Supply 2 km ductile iron and HDPE

	Install 2 km ductile iron and HDPE
Antrim WTP	Upgrade capacity by 470 m^3/hr
Monitoring	Leakage detection equipment
	Bulk meters
	Real time SCADA (supervisory control and data acquisition)
Water intake	New intake on Check Hall River
	Upgrade existing intake at Springfield

Table 4-26. Dominica water infrastructure improvement project (CDB, 2020)

In Trinidad and Tobago, cost estimates for improvements to water infrastructure have been studied under various scenarios. Excerpts from this analysis are presented in Table 4-27 (GEF-CReW, 2017).

	Cost, US$M[9]	
Component	**Existing Condition**	**Improved Condition**
Annual capital expense	0.2 per plant	N/A
Annual operating expense	0.1 per plant	N/A
Periodic upgrade (5-year interval)	0.4	N/A
Capital cost	N/A	22 (total) including 0.8 engineering design 3.3 land management
Indirect costs	Needs exceed capacity; Not meeting water quality	Additional service; Meet water quality; Lower operation and maintenance costs

Table 4-27. Wastewater improvement in Trinidad and Tobago (adapted from GEF-CreW, 2017)

4.8.3 Strengthening costs

In Trinidad and Tobago, trenchless technologies have been examined for strengthening of underground pipes. Table 4-28 (CWWA, 2017) presents the cost estimates for upgrading for various pipe diameters.

Pipe size, mm	**Conventional, US$/m**	**Trenchless, US$/m**
300	275	50-175
500	400	200-500
1200	800	500-1000
1800	1500	850-1600
2500	3000	1500-3000

Table 4-28. Example of underground pipe strengthening cost (CWWA, 2015)

Eidinger (2012) examined the economic benefits of seismic retrofitting of water infrastructure; see Table 4-29 and Table 4-30. The data is shown for the municipalities in the eastern San Francisco Bay Area. The level of earthquake considered in this study (magnitude 7.1) could be representative of earthquakes considered in some subject countries, such as Haiti or Trinidad and Tobago. As shown in Table 4-30, the cost associated with retrofitting is less than the expected economic loss cost for the scenario event.

Parameters	**Assets**
Population served	1,200,000
Transmission pipes	320 km
Distribution pipes	4,600 km
WTP	6

[9] Based on a rate of 0.15 US$ to 1 TT$

Storage tanks	175
Pump stations	125

Table 4-29. Water infrastructure components in the study (adapted from Eidinger, 2012)

Parameters	Existing	Upgraded
Pipe repairs, scenario earthquake	3,300 to 5,000	--
Seismic upgrade cost	--	US$240 M
Unit cost	--	$200 per person
Economic loss	US$680 M	US$410 M
Savings		US$270 M
Δ= Benefits-cost		US$30 M
Unit Δ		$25 per person

Table 4-30. Cost and benefit associated with strengthening of water infrastructure (adapted from Eidinger, 2012)

MWH Global, Inc. (2012) developed cost estimates for retrofitting of a WTP in the U.S. state of Oregon; see Figure 4-37. The original construction in the 1930s included the headhouse, filters, flocculation and sedimentation. In the 1950s and 60s, chemical storage tanks and additional basins were added and in the 1980s, a water intake structure was added.

Figure 4-37. Existing WTP in Oregon (MWH, 2012)

Analysis by MWH Global, Inc. (2012) indicated that seismic and landslide hazards were the main natural hazards for the site and a number of vulnerable components were identified:

- Sedimentation basin walls were thin and subject to hydrodynamic forces.
- Original buildings used weak and non-ductile details.
- Filters used thin concrete walls.
- Chemical tanks were not detailed for adequate level of force.

To mitigate vulnerabilities, retrofit options were proposed and costs estimates were developed; see Table 4-31.

Component	Cost, US$M
Building	1.8
Basin walls	2.0
Tanks	0.8
Pipe anchorage	0.2
Equipment anchorage	0.2

Total	**5.0**

Table 4-31. Estimated retrofitting cost for WTP in Oregon (adapted from MWH, 2012)

The authors also examined the reconstruction cost for a new plant if the damage to the existing WTP necessitated building a new plant; see Table 4-32. The associated estimates for piping are presented in Table 4-33.

Component	Cost, US$M
Buildings	3.0
Basins	4.0
Tank and storage	4.5
Filters	7.5
Pump stations	4.0
Total	**23.0**

Table 4-32. Estimated construction cost for WTP in Oregon (adapted from MWH, 2012)

Diameter, m	Cost, US$/m
0.5	1,600
0.6	2,000
0.75	2,500
0.9	3,000
1.2	4,000

Table 4-33. Estimated construction cost for pipes (adapted from MWH, 2012)

It is noted that retrofitting (strengthening) is very cost effective when compared to new construction.

4.8.4 Life-cycle cost

For water infrastructure, life-cycle cost analysis (LCCA) is typically used in design. This topic will be further discussed in Chapter 8.

4.9 Discussion

In the subject Caribbean countries, there are a large number of water infrastructures, including WTP, WWTP, and many kilometers of pipelines. The courtiers are also located in one of the most hazard-prone areas in the world, experiencing a number of hurricanes annually and earthquakes periodically. These natural hazards have caused significant damage to water infrastructure in recent years, causing economic distresses and delayed recovery. Accordingly, it is time-critical to plan and implement measures that result in resilient water infrastructure. Experience has shown that for a given sub-class of water infrastructure, there are a number of components that are the most susceptible to damage in natural hazard events. For these components, effective retrofitting and mitigation methods are available that reduce the expected damage to infrastructure and are cost effective. Table 4-34 presents a summary of findings related to the most critical components of water infrastructure in the subject countries.

Subsector	Hazard	Vulnerable Component	Resilience Measure	Cost, % of initial cost (Est.)	
				Improvement cost	**Vulnerability reduction**
WTP/WWTP: Plant building (Masonry/RC frame), Water storage, Equipment	Earthquake	Wall out-of-plane (Building)	Add wall bracing (anchor and strongback)	15%	40%
		In-plane (Building)	Add RC shear wall	20%	60%
		Ductility (Building)	Add column jacketing	10%	30%
		Foundation (Water storage)	Add concrete foundation	40%	50%
		Anchorage (Water storage)	Strengthen anchorage	10%	30%
		Foundation tie (Equipment)	Improve foundation tie	10%	20%

	Liquefaction	Foundation (All)	Enlarge spread footing or add pile foundation	40%	80%
		Soil (All)	Install soil grouting	30%	80%
	Wind	Roof connection (Building)	Strengthen roof connection	10%	50%
		External wall (Building)	Improve wall connection	10%	30%
		Foundation tie (Equipment)	Improve foundation tie	10%	20%
	Flood	Building envelope (Building)	Install flood shield	10%	80%
		Elevated construction (Building)	Construct floodwall	15%	80%
		Elevating (Equipment)	Elevate equipment	20%	80%
		Dry floodproofing (Equipment)	Install watertight barrier	15%	80%
Underground pipelines (UGP)	Earthquake, Liquefaction	Pipes	Replace with ductile pipes, Trenchless technologies	40%	80%
		Joints	Add flexible joints	30%	80%

Table 4-34. Summary of findings for water infrastructure in the Caribbean

5. TRANSPORT INFRASTRUCTURE

5.1 Introduction

Improving the resiliency of transport infrastructure has a multi-fold beneficial impact for a country: by reducing physical damage to infrastructure components, economic losses are reduced, and the continuous operation of roads and bridges allows for planners to direct resources and emergency personnel to the most adversely affected areas to assist in recovery. Bridges in particular are a critical infrastructure lifeline, as damage to them will sever the link between connected communities and will require use of detours that can then add time to commuting and recovery. Accordingly, bridges are emphasized in this chapter.

For transport infrastructure in the Caribbean, the governing hazards and infrastructure components listed in Table 5-1 will be used.

Sector	Subsector	Type	Components	Hazard
TRANSPORT	Paved Roads	Asphalt paved	Subsurface	Flood
			Surface	Flood
	Bridges	Concrete girder	Columns	Earthquake
			Joints	Earthquake
		Steel truss	Members	Earthquake
			Connections	Earthquake
		Steel girder	Bearings	Earthquake
			Cross frames	Earthquake
		Steel bridge	Welded connections	Wind
		Suspension	Bridge Stability	Wind
		All	Foundation	Earthquake

Table 5-1. Analysis matrix for transport infrastructure

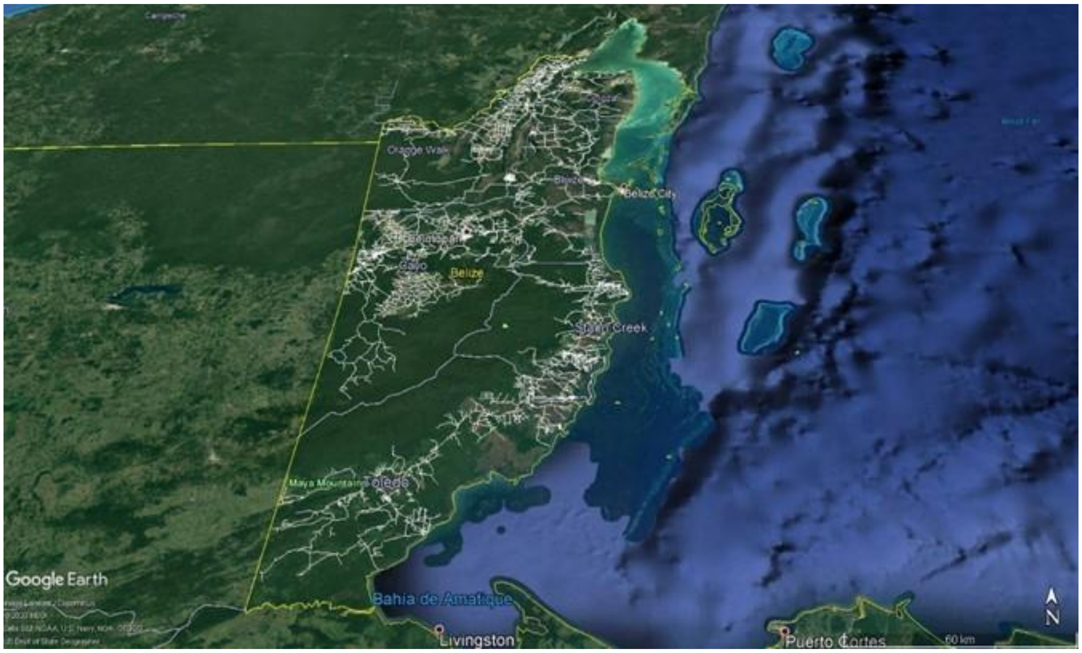
5.2 Geographical distribution

In the subject countries, the following government entities are responsible for construction, maintenance, and upgrading of the transport sector, including road and bridges; see Table 5-2.

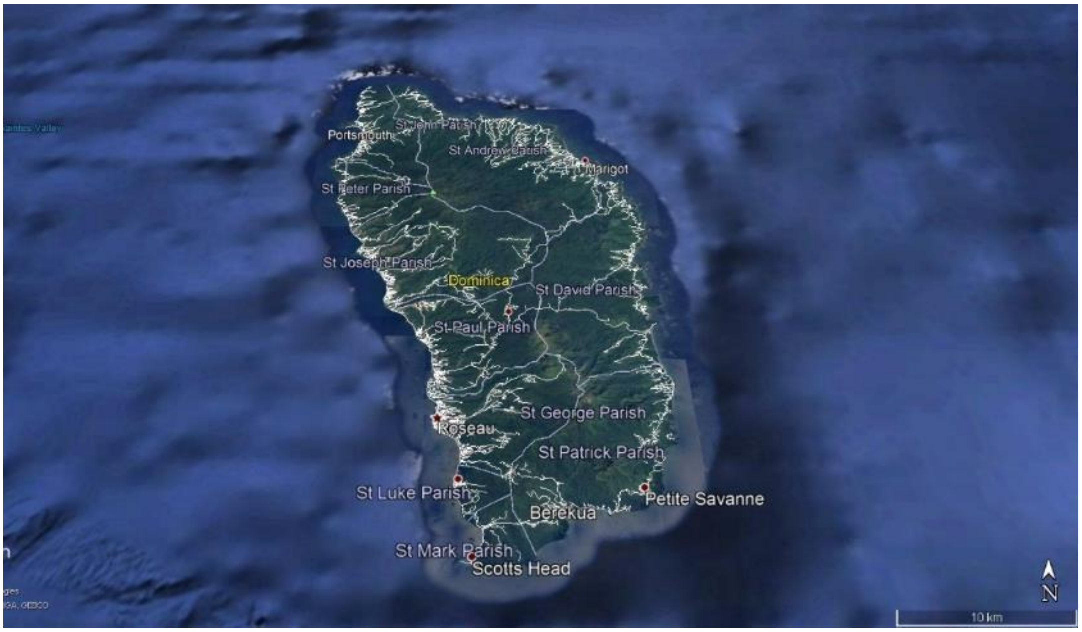
Country	Agency
Suriname	Ministry of Transport, Communication and Tourism (MINTCT)
Trinidad and Tobago	Ministry of Works and Transport (MOWT)
Guyana	Ministry of Public Infrastructure (MOPW)
Belize	Department of Transport (DOT)
Haiti	Ministry of Public Works, Transportation and Communications
Dominican Republic	Ministry of Public Works and Communications
Antigua and Barbuda	Ministry of Public Utilities, Civil Aviation and Energy
Dominica	Ministry of Public Works and The Digital Economy
Grenada	Ministry of Infrastructure Development, Public Utilities, Energy, Transport & Implementation
Saint Kitts and Nevis	Ministry of Public Works, Utilities, Transport and Postal Services
Saint Lucia	Ministry of Infrastructure, Ports, Energy and Labour (MIPST)
Saint Vincent and the Grenadines	Ministry of Transport, Works, Urban Development and Local Government
Sint Maarten	Ministry of Tourism, Economic Affairs, Transport and Telecommunication
Barbados	Ministry of Transport, Works and Maintenance
Jamaica	Ministry of Transport and Mining (MTM)
Bahamas	Ministry of Transport and Local Government

Table 5-2. Transport infrastructure authorities for 16 subject countries

Figure 5-1 presents distributions of road networks in Belize, Dominica, Grenada, Saint Lucia, Suriname, and Saint Vincent and the Grenadines. Note that both primary and secondary (including unpaved roads) are shown. As seen in the figures, most roads are located close to shorelines and as such, could be vulnerable to flooding in the aftermath of hurricanes.



Belize



Dominica

Grenada

Saint Lucia

Suriname

Saint Vincent and the Grenadines

Figure 5-1. Road networks (various sources)

5.3 Typology

For bridges, the type usually refers to the superstructure construction. Examples of bridge types include girder bridges, truss bridges, arch bridges, and suspension bridges. As discussed earlier, this study focuses on existing bridges constructed prior to the year 2000. It is assumed that new bridges or bridges currently in design will have a better expected performance than bridges of older vintages. Examples of bridges in the study area are presented in Figure 5-2

For paved roads, the surface course is either bituminous, which is regarded as flexible pavement, or concrete, which is rigid; see Figure 5-3. Concrete paved roads have a better performance than asphalt roads; in the Caribbean, most roadways are asphalt. Thus, asphalt roads are presented in this report.

Concrete girder bridge, Trinidad and Tobago

Concrete girder bridge, Haiti

Steel girder bridge, Suriname

Steel girder bridge, Dominican Republic

Steel truss bridge, Guyana

Steel truss bridge, Belize

Suspension bridge, Trinidad and Tobago | Suspension bridge, Belize

Figure 5-2. Bridge typology (various sources)

Asphalt pavement, Dominica | Concrete pavement, Jamaica

Figure 5-3. Paved road typology (various sources)

5.4 Vulnerable components

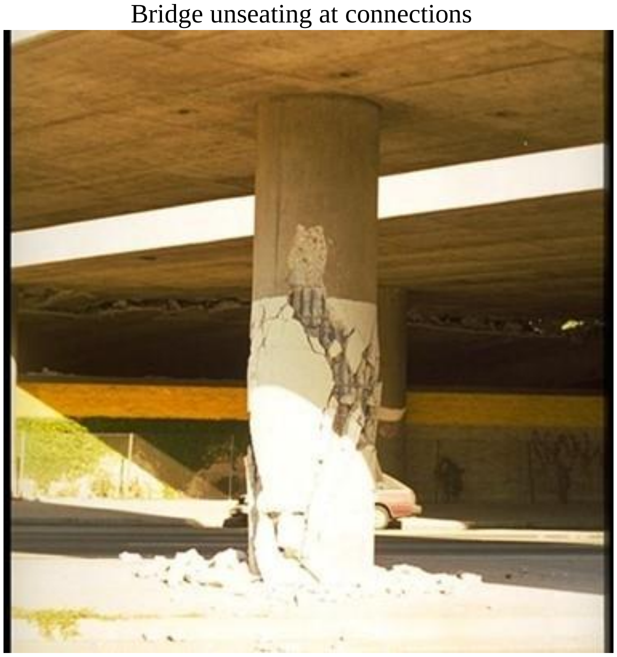
Past natural hazard events have shown that certain components of roads and bridges are vulnerable to various natural hazards. Certain components have been known to experience frequent failures. Strengthening of these components would lead to marked improvement in performance and as such, these components are emphasized in this chapter. Examples of earthquake damage to vulnerable components of bridges are presented in Figure 5-4

Girder bridges can be vulnerable to hurricane wind forces and storm surge flooding. Simply-supported girder bridges can fail if the supports (bearings) have inadequate capacity to accommodate displacements imposed by wind forces. Suspension bridges are flexible and thus are not expected to experience damage in earthquakes. However, if they lack strength or stiffness, they can be vulnerable to wind forces from hurricanes. Examples of hurricane damage to vulnerable components of bridges are presented in Figure 5-5 There have been instances of floods washing bridges away.

Paved roads are most vulnerable to flooding, and in particular to hurricane storm surge flooding, which unleashes large volumes of water at high velocity that can wash away the road, surface, and subsurface. Examples of flood damage to roadways are presented in Figure 5-6.

Bridge unseating at connections

Concrete column failure due to lack of confinement



Column shear failure

Buckling of slender members

Figure 5-4. Bridge damage in California earthquakes (NISEE, 2020)

Dropped spans (USGS, 2007)

Structural instability (various)

Fatigue cracking (Haghani et al., 2012) Damaged bridge (Floodlist, 2015)

Figure 5-5. Bridge damage as a result of wind forces and associated flooding

Washed away road (The Jamaican, 2017) Damaged road (Floodlist, 2016)

Figure 5-6. Road damage due to flooding

Table 5-3 presents the components that have experienced the most damage in past events and that will be considered in this chapter.

Subsector	Type	Component	Vulnerability	Hazard
Paved roads	Asphalt paved	Subsurface	Substrate washout	Flood
		Surface	Strength loss	
Bridges	Concrete girder	Columns	Lack of ductility	Earthquake
		Joints	Short seat	
	Steel truss	Members	Slender members	
		Connections	Riveted connections	
	Steel girder	Bearings	Unseating	
		Cross frames	Slender members	
	Steel bridges	Welded connections	Low fatigue capacity	Wind
	Suspension bridge	Bridge Stability	Low torsional stiffness	Wind
	All	Foundation	Lack of capacity	Liquefaction

Table 5-3. Vulnerable components of transport infrastructure and corresponding hazards

5.5 Bridges

5.5.1 Design codes

5.5.1.1 Seismic design in Trinidad and Tobago

In Trinidad and Tobago, a multi-year project of bridge assessment and reconstruction was completed in the early 2010s (Khan-Kernahan, 2013). As part of this project, 96 bridges underwent rapid assessment. From that pool, 70 bridges were selected for detailed assessment, 40 of which were selected for reconstruction. These reconstructed bridges were designed by international engineering consultants and the bridge design followed the American Association of Highway and Transportation Officials (AASHTO) design procedures in effect at the time. Thus, it is expected that such bridges incorporated seismic measures. At this time, the design procedures for seismic loading for other bridges in the region are not known.

The 975-year seismic map is presented in Figure 5-7 This earthquake return period is usually used for bridges. As it can be seen in Figure 5-7, the bedrock acceleration is as high as 0.5g . For design, the bedrock values need to be multiplied by site coefficients to account for the soil flexibility.

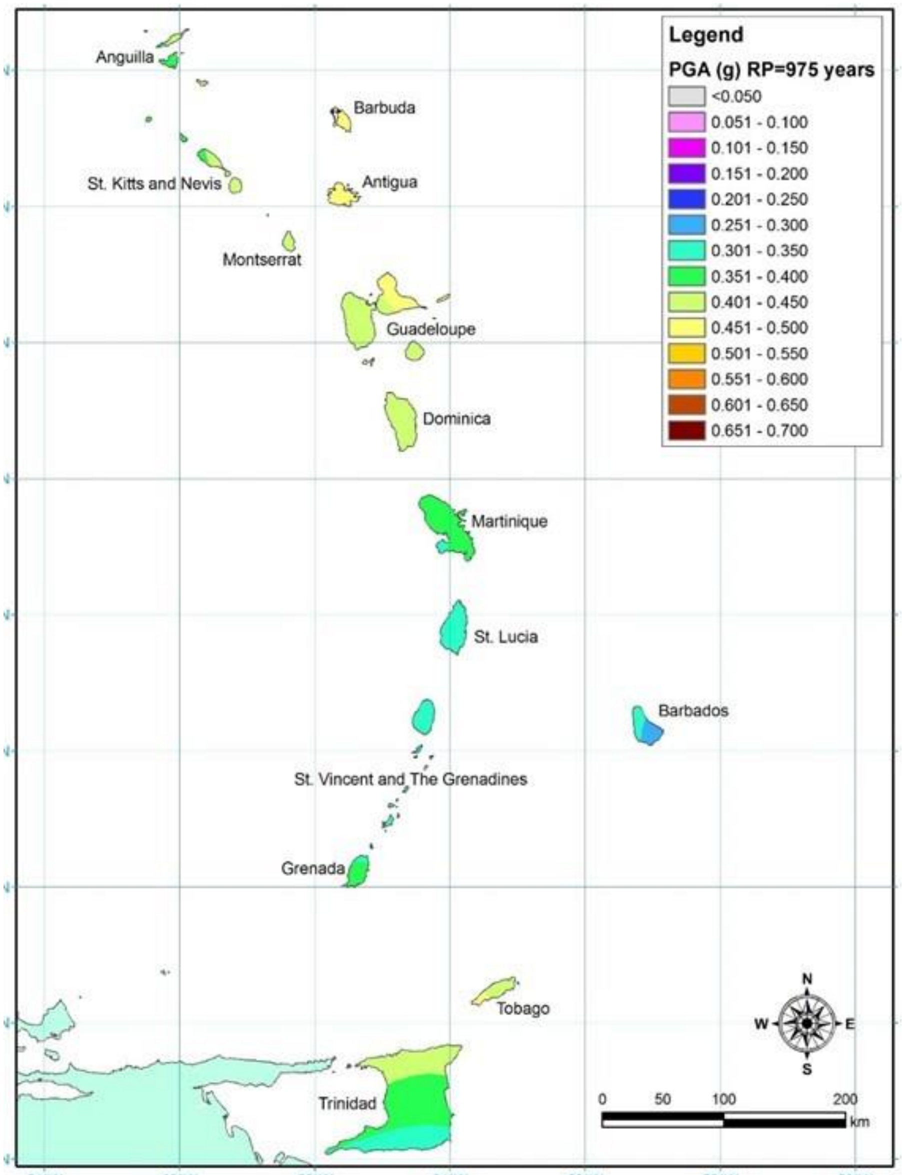


Figure 5-7. PGA for 975-year earthquake (UWI, 2020)

5.5.1.2 Wind design in the Dominican Republic

For the Dominican Republic, the wind design is based on the *Manual for Wind Design* (Grupo Estabilidad Estructural, 2000). The country is divided into three zones for wind loading; see Figure 5-8. For Zones I through III, the basic design wind speed is 240, 210, and 180 km/h, respectively. The key components of wind design for structures are as follows:

- Design procedure based on US approach; wind pressure is computed based on the wind speed.
- An importance factor is assigned to the structure
- Internal and external wind coefficients are computed
- Gust effects are taken into account
- Evaluation is based on analytical procedure or wind tunnel testing

It is anticipated that the provisions for wind design would vary depending on the level of expected hurricane hazard in subject countries. For example, in countries that experience frequent hurricanes, such as The Bahamas and Leeward Islands, wind loading provisions would be more important compared to the subject counties in South America.

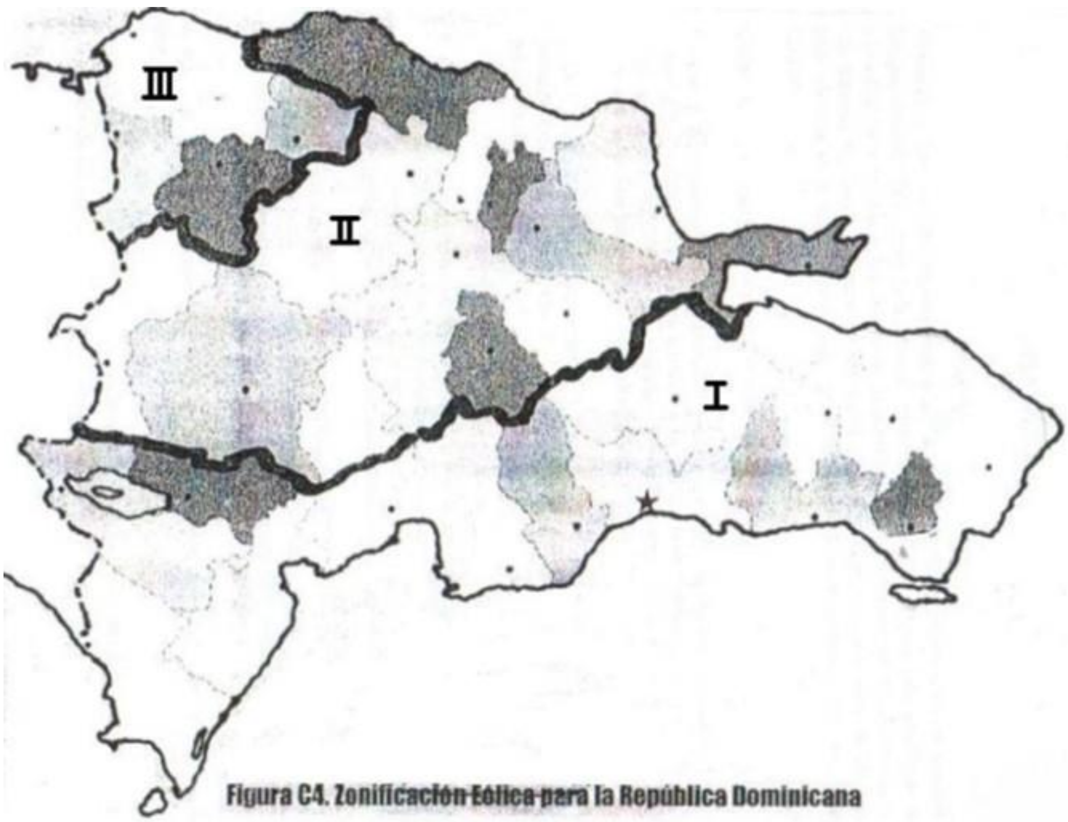


Figure 5-8. Wind zones in the country (Grupo Estabilidad Estructural, 2000)

5.5.2 Fragility functions based on FEMA Hazus methodology

5.5.2.1 Overview

In this section, physical (structural) damage and operation loss (functionality) fragility relations for bridge types common in the Caribbean are presented. For the bridge structures, two types of design are considered: conventional (non-seismic design constructed prior to adoption of seismic codes) and seismic (constructed after adoption of seismic provisions in bridge design). Federal Emergency Management Agency (FEMA)

Hazus (FEMA, 2003a) defines a number of motor (highway) bridge types. The typologies relevant to the Caribbean for multi-column bents are summarized in Table 5-4.

Typology	Superstructure material	Superstructure type	Continuity	Seismic design
HWB5	Concrete	Slab	Simply supported	No
HWB7		I girder		Yes
HWB10		T beam	Continuous	No
HWB11		Box girder		Yes
HWB12	Steel	Beam	Simply supported	No
HWB14		Stringer and floor beam		Yes
HWB15		Plate girder	Continuous	No
HWB16		Truss		Yes

Table 5-4. Bridge typologies (adapted from FEMA, 2003a)

5.5.2.2 Damage states

FEMA Hazus (FEMA, 2003a) defines a number of damage states for bridges. The damage states relevant to the Caribbean are summarized in Table 5-5.

Damage state	State	Description
DS1	None	No observable damage
DS2	Slight	Minor cracking and spalling to the abutment, cracks in shear keys at abutments, minor spalling and cracks at hinges, minor spalling at the column (damage requires no more than cosmetic repair) or minor cracking to the deck
DS3	Moderate	Any column experiencing moderate (shear cracks) cracking and spalling (column structurally still sound), moderate movement of the abutment (<50 mm), extensive cracking and spalling of shear keys, any connection having cracked shear keys or bent bolts, keeper bar failure without unseating, rocker bearing failure or moderate settlement of the approach.
DS4	Extensive	Any column degrading without collapse – shear failure - (column structurally unsafe), significant residual movement at connections, or major settlement approach, vertical offset of the abutment, differential settlement at connections, shear key failure at abutments.
DS5	Complete	Any column collapsing and connection losing all bearing support, which may lead to imminent deck collapse, tilting of substructure due to foundation failure.

Table 5-5. Bridge damage states (adapted from FEMA, 2003a)

5.5.2.3 Fragility functions for ground shaking

Based on the bridge types and damage states, FEMA Hazus (FEMA, 2003a) has developed fragility functions for bridges corresponding to the probability of exceeding a damage state. Fragility functions relevant to common bridges in the Caribbean are summarized in Table 5-6.

	Mean, S_a (1.0 sec), g				Standard deviation, g
Typology	DS2	DS3	DS4	DS5	
HWB5	0.25	0.35	0.45	0.7	0.6
HWB7	0.5	0.8	1.1	1.7	0.6
HWB10	0.6	0.8	1.1	1.5	0.6
HWB11	0.9	0.8	1.1	1.5	0.6
HWB12	0.25	0.35	0.45	0.7	0.6
HWB14	0.5	0.8	1.1	1.7	0.6
HWB15	0.75	0.75	0.75	1.1	0.6
HWB16	0.9	0.9	1.1	1.5	0.6

Table 5-6. Baseline bridge fragility parameters, ground motion (adapted from FEMA, 2003a)

Note the following:

- For simply supported bridges, seismic design has significant differences (see for example HWB5 and HWB7). This is because failure (unseating) at in-span hinges is a key mode of failure. For continuous bridges, in-span hinges are not present and there is more redundancy because of framing action in the longitudinal (along bridge centerline) axis.
- Performance of steel and concrete bridges are similar. This is especially true when seismic measures are incorporated in design.
- It is anticipated that a majority of bridges in the Caribbean are not continuous, that is, the superstructure is placed on joints above pier caps.

The baseline mean values need to be modified for specific bridge configuration and geometry. The modification factors comprise the following:

- Skew factor. Bridge skew introduces coupled and torsional response in the bridge. The skew is represented by angle α which is the angle between the centerline of bridge and centerline of piers (bents).

$$\text{Eq. 5-1 } K_{skew} = \sqrt{\sin{(90 - \alpha)}}$$

- Geometry factor. A factor that accounts for the number of spans (N) in a bridge. Parameters A and B are functions of the bridge type.

$$\text{Eq. 5-2} K_{geo} = 1 + \frac{A}{N-B}$$

5.5.2.4 Fragility functions: ground failure (liquefaction)

The fragility functions for ground failure hazards are presented in this section. These functions are presented in terms of permanent ground displacement (PGD).

Based on the bridge types and damage states, FEMA Hazus (FEMA, 2003a) has developed fragility functions for bridges corresponding to the probability of exceeding a damage state. Fragility functions relevant to common bridges in the Caribbean are summarized in Table 5-7.

	Mean, PGD, mm				Standard deviation, mm
Typology	**DS2**	**DS3**	**DS4**	**DS5**	
HWB5	90	90	90	350	5
HWB7	90	90	90	350	5
HWB10	90	90	90	350	5
HWB11	90	90	90	350	5
HWB12	90	90	90	350	5
HWB14	90	90	90	350	5
HWB15	90	90	90	350	5
HWB16	90	90	90	350	5

Table 5-7. Baseline bridge fragility parameters, ground failure (adapted from FEMA, 2003a)

Note the following:

- The baseline fragility parameters are independent of the bridge type, as damage is caused by the underlying soil.

The baseline mean values need to be modified for specific bridge configuration and geometry. The modification factors comprise the following:

- Skew factor. Bridge skew introduces coupled and torsional response in the bridge. The skew is represented by angle α which is the angle between the centerline of the bridge and centerline of piers (bents).

Eq. 5-3 $f_1 = \sin(\alpha)$

- Geometry factor. A factor that accounts for the number of spans (N), bridge width (W) and bridge length (L).

Eq. 5-4 $f\_2 = 0.5L/(NWsin(\alpha))$

5.5.2.5 Example Bridge

To assess the effect of seismic strengthening measures on a bridge, consider the bridge listed in Table 5-8, which is a typical bridge in the Caribbean.

Case	Superstructure	Continuity	Substructure	Geometry	Liquefiable soil	Condition
1A	Concrete I girder	Simply supported	Two column piers	50 m long, 16 m wide, 30° skew	yes	As-is
1B						Retrofitted

Table 5-8. Properties of the example bridge

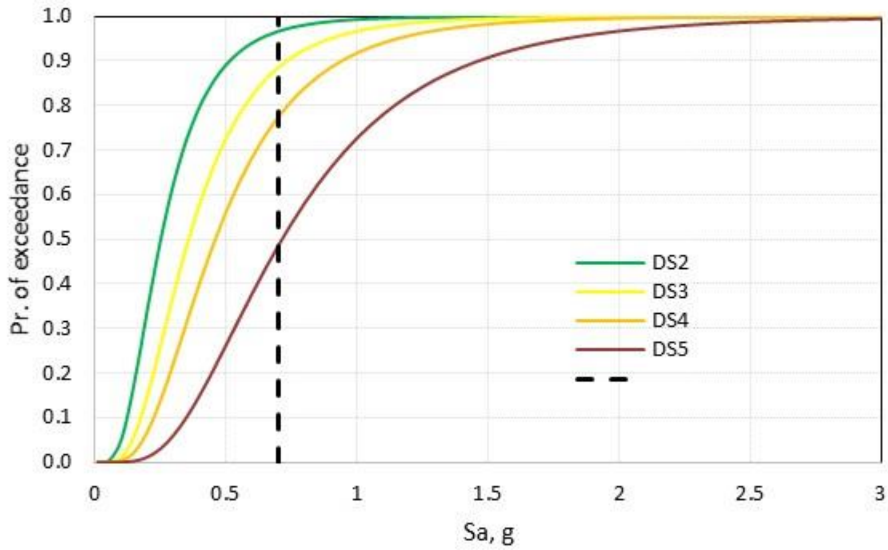
The modification factor for fragility functions for these two cases above has been computed; see Table 5-9. Also shown in Table 5-9 are the computed modified fragility parameters.

	Type	Existing	Retrofitted
	A	0.25	0.25
	B	1	1
	K_{skew}	0.9	0.9
	K_{geo}	1.1	1.1
	f1	0.6	0.6
	f2	0.9	0.9
Mean, g	DS2	0.25	0.5
	DS3	0.43	0.98
	DS4	0.46	1.12
	DS5	0.71	1.73
Mean, mm	DS2	80	80
	DS3	80	80
	DS4	80	80
	DS5	310	310

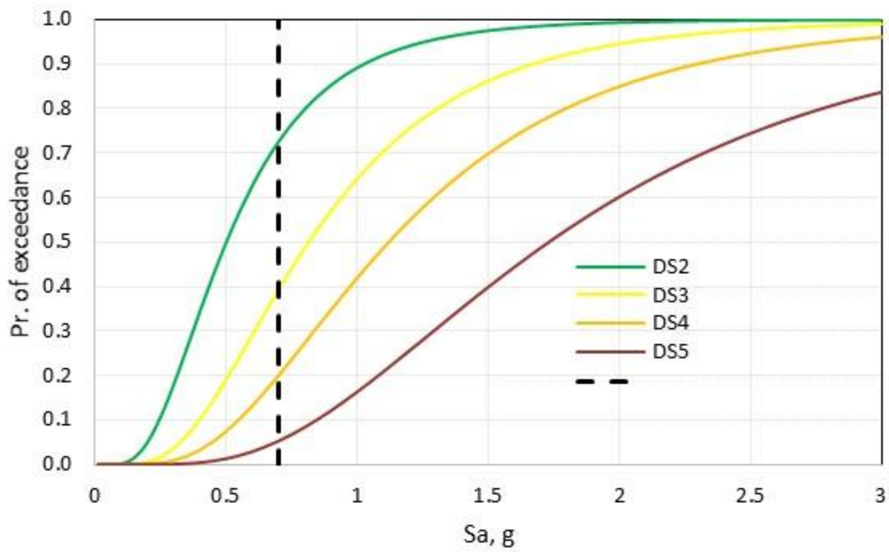
Table 5-9. Modifications for fragility functions

Figure 5-9 presents the fragility function for the example bridge in Table 5-8. Also shown in the figure is the line with an ordinate of approximately 0.75 g, which corresponds to the 1-sec spectral acceleration that could be typical for Trinidad and Tobago, Haiti, or the Dominican Republic. As seen in the figures, the probability of experiencing significant damage is reduced measurably when retrofit measures are implemented.

Figure 5-10 presents the probability of the bridge being in a given damage state when subjected to an expected (design) earthquake. Note that once seismic retrofitting is implemented, the probability of the bridge experiencing the detrimental DS4 or DS5 damage states is reduced from approximately 80% to approximately 20%.



Existing



Retrofitted

Figure 5-9. Fragility functions for example bridge

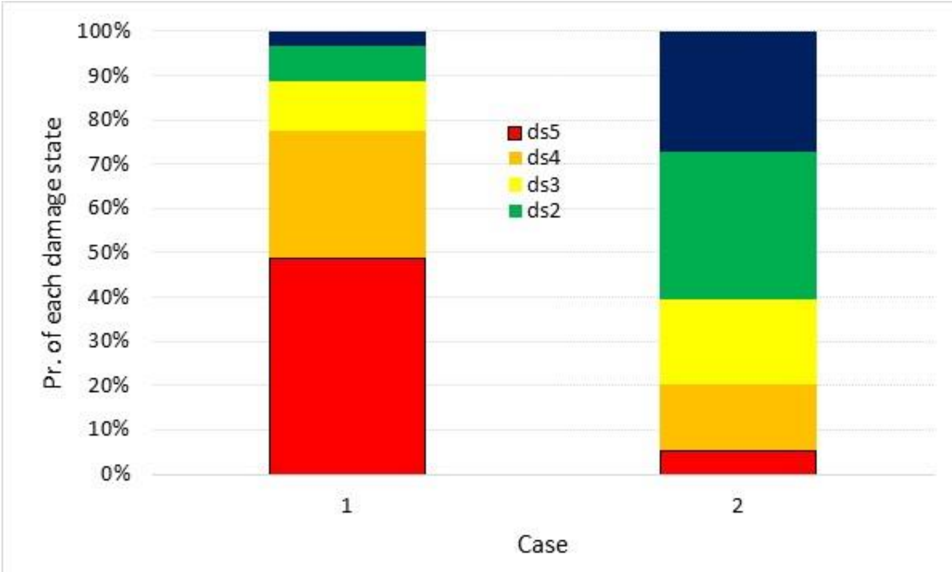


Figure 5-10. Distribution of damage states

The mitigation for liquefaction consists of soil improvements or addition of deep foundations. Once such measures are implemented, the ground failure hazard is essentially eliminated; see Figure 5-11.

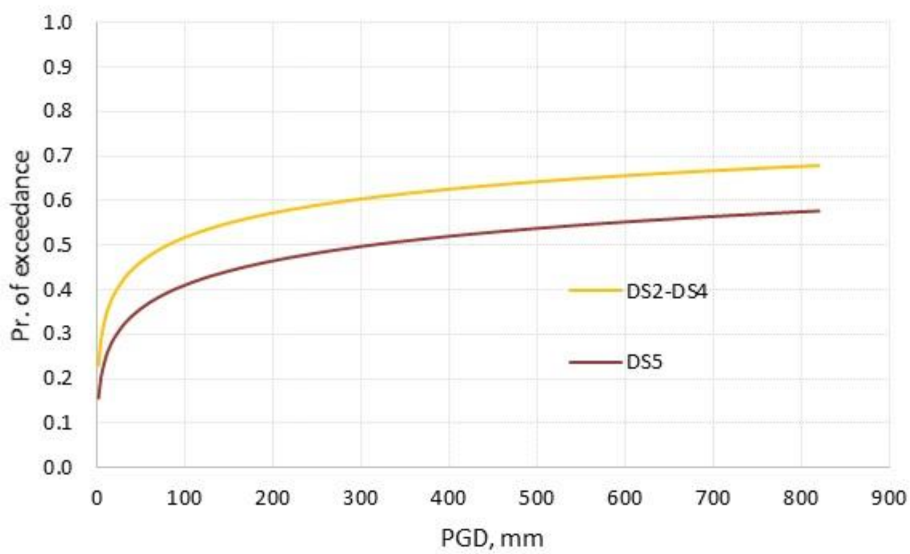


Figure 5-11. Ground failure (liquefaction), existing case

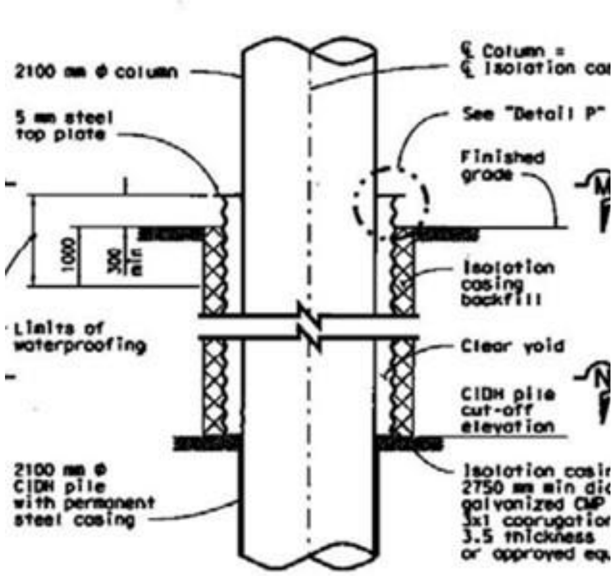
5.5.3 Strengthening measures for earthquakes

This section provides examples of seismic retrofits for the potential expected deficiencies in the bridge types considered in this report. The subsequent sections provide information on the cost and benefits of such strengthening measures.

5.5.3.1 Isolation casing

When columns in a bent, bents in a frame, or frames in a bridge have fundamental vibration periods that differ by more than 10-20%, during the seismic events, the earthquake (inertial) forces are attracted disproportionally to the stiffer members and could cause severe damage to these members.

Isolation casings are an inexpensive and effective method to mitigate unbalanced configuration by increasing the column length and ensuring a more uniform distribution of seismic forces to the bridge components; see Figure 5-12.

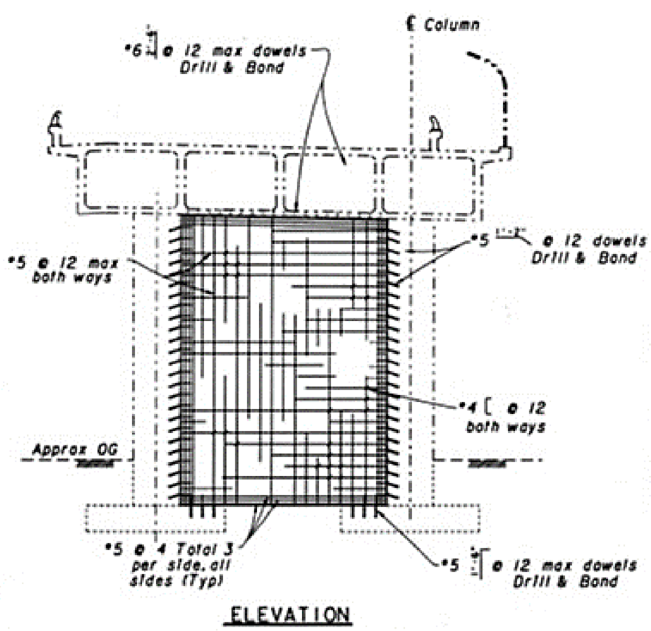


Typical plans Retrofitted bridge

Figure 5-12. Addition of isolation casing (Caltrans, 2020)

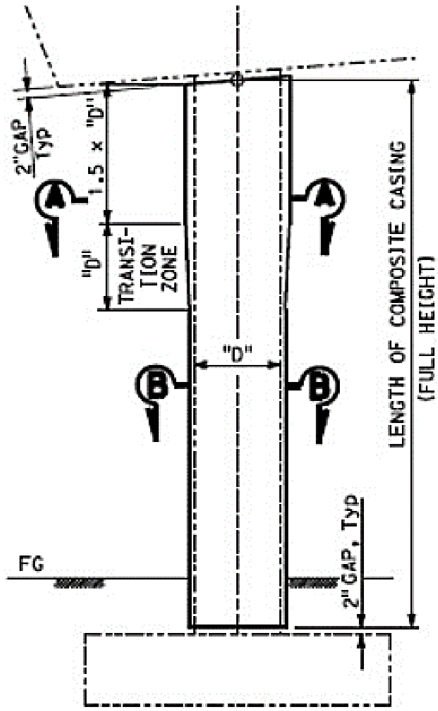
5.5.3.2 Concrete column retrofit for increased capacity

Column failure in earthquakes could result in collapse of the bridge. When columns have inadequate shear or flexural capacity, pier walls can be added; see Figure 5-13. The walls serve to carry most of the seismic force and thus reduce the demand on the existing columns. For columns with poor confinement or inadequate confinement, Fiber-Reinforced Polymer (FRP) can be added to rectangular or circular columns to enhance performance; see Figure 5-14.



Typical plans Retrofitted bridge

Figure 5-13. Addition of pier walls (Caltrans, 2020)



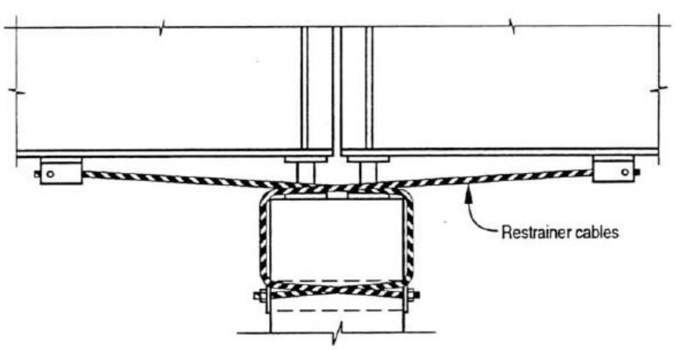
Typical plans Retrofitted bridge

Figure 5-14. Addition of column casings (Caltrans, 2020)

5.5.3.3 Retrofit for continuity

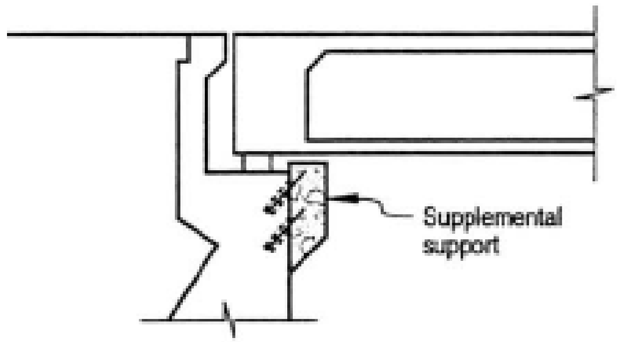
The unearthing of the superstructure at joints (hinges) is a common failure for existing bridges and occurs because the hinge or bearings do not have sufficient length. For retrofit of steel or concrete girder bridges supported on bearings at top of piers, cable restrainers are used to prevent excessive movement; see Figure 5-15. This still allows for thermal movement of the bridge, but prevents unseating at large displacement.

Unseating could also occur at bridge abutment when the superstructure in supported on bearings and bridge travel exceeds the provided seat width. This is especially critical for bridges at large skews. The retrofit comprises constructing reinforced concrete bolsters in front of the abutment to extend the travel length; see Figure 5-16.



Typical plans Retrofitted bridge

Figure 5-15. Addition of cable restrainers at piers (Caltrans, 2020)

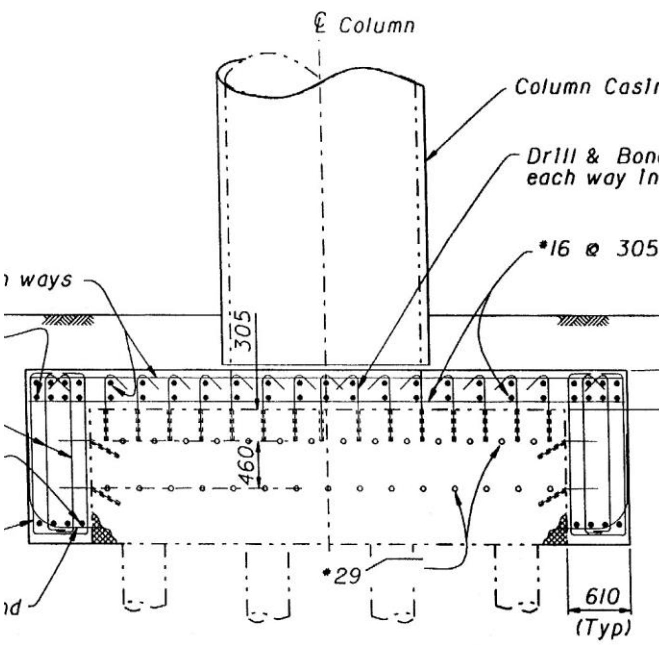


Typical plans Retrofitted bridge

Figure 5-16. Retrofit of abutment seats (Caltrans, 2020)

5.5.3.4 Foundation retrofit

When the foundation has inadequate capacity, retrofitting consists of enlarging the foundation footprint and likely adding new piles; see Figure 5-17. Steel driven or miscopies can be used. Foundation retrofitting is typically expensive.

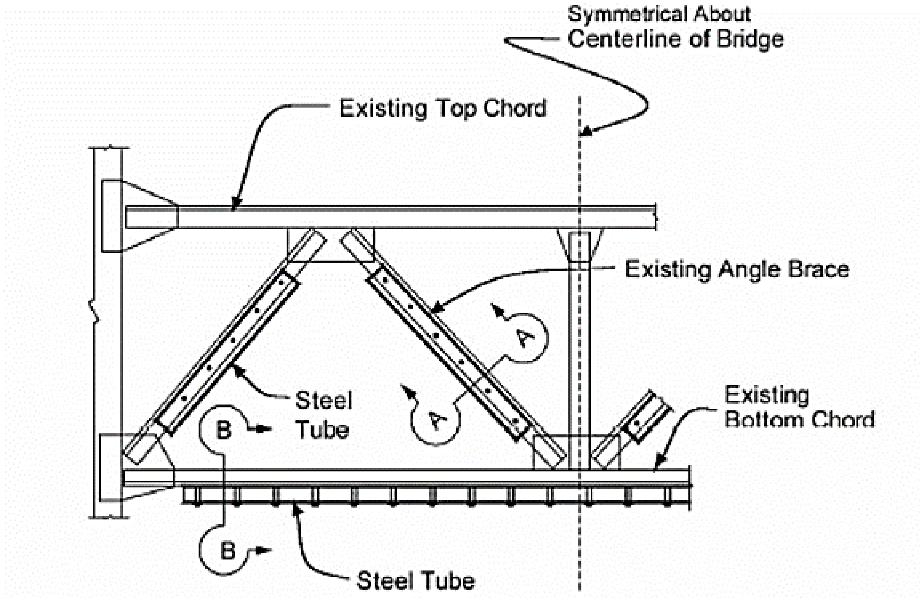


Typical plans Retrofitted bridge

Figure 5-17. Retrofit of foundations (Caltrans, 2020)

5.5.3.5 Retrofit of steel diaphragms for steel girder bridges

For many existing steel girder bridges, diaphragms (cross frames) are constructed using slender steel angles or double angles. Tube steel can be added to increase stiffness and capacity of diaphragms; see Figure 5-18.

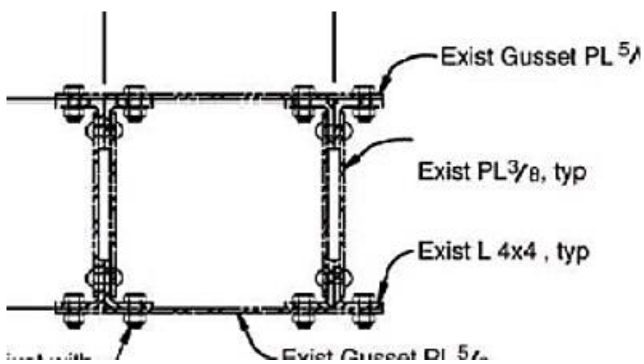


Typical plans

Figure 5-18. Retrofit of steel girder diaphragms (FHWA, 2006)

5.5.3.6 Retrofit of riveted members

Existing steel truss or girder bridges constructed before the 1960s typically used riveted members or splices, respectively. Riveted attachments are not as robust as high-strength bolted connections. These members can be retrofitted by replacing the rivets with high-strength bolts; see Figure 5-19.

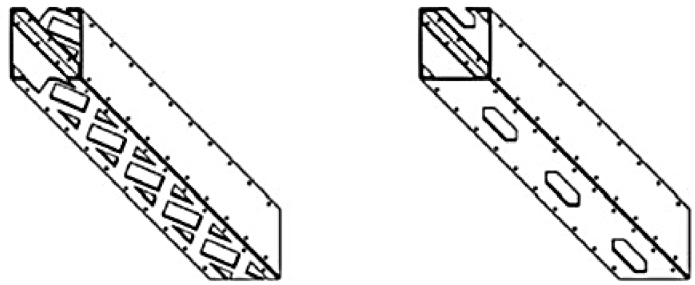


Typical plans

Figure 5-19. Retrofit of steel truss bridge members by replacing rivets with high-strength bolts (Ho, 2006)

5.5.3.7 Retrofit of lattice members

Many existing steel truss bridges use lattice members. This was done to reduce material cost during construction. However, lattice members are more flexible than rolled beams or built-up girder and more susceptible to global or local buckling. Retrofit of these members consists of adding cover plates to the existing channels that have been originally connected by plate lattices; see Figure 5-20.

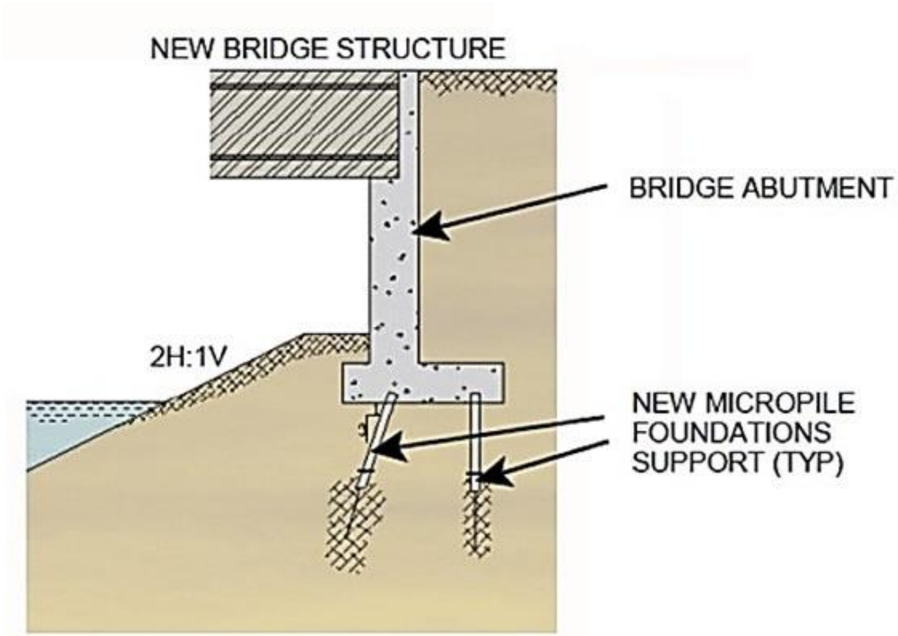


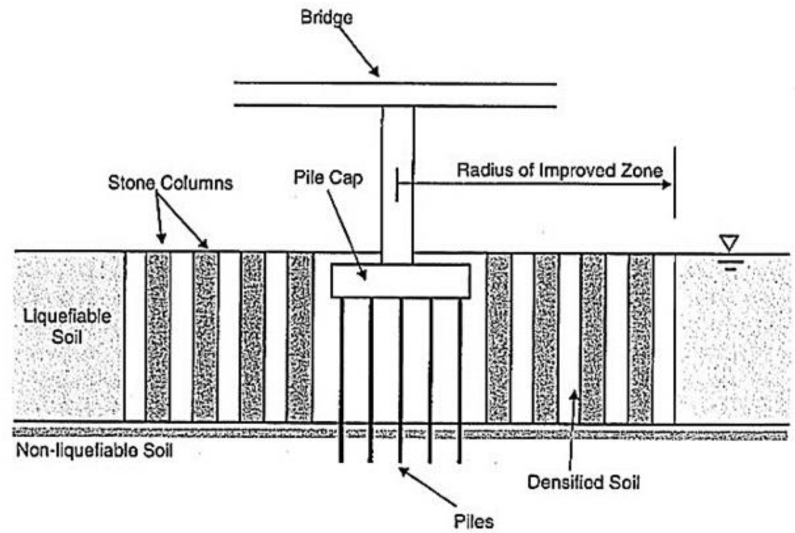
Typical plans

Figure 5-20. Retrofit of steel riveted lattice members by cover plates (Ho, 2006)

5.5.4 Strengthening measures for liquefaction

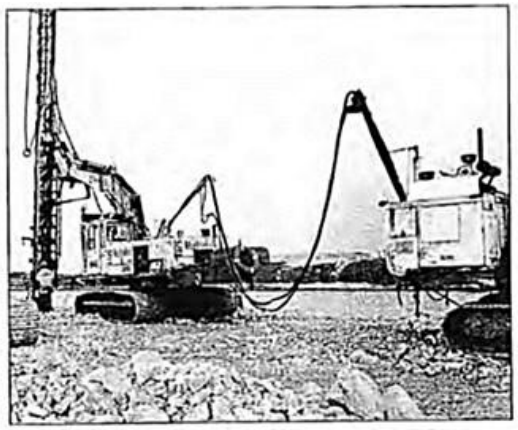
For sandy soils located in areas of high ground water elevation, liquefaction is a risk when subject to earthquake hazard. Liquefaction is more likely to occur when earthquakes have a larger magnitude. Examples of ground improvements to mitigate liquefaction are presented in Figure 5-21.



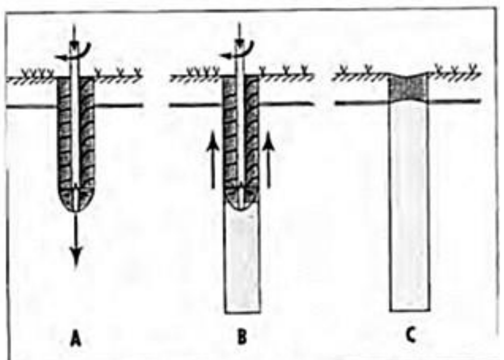


Add new mico piles

Densification

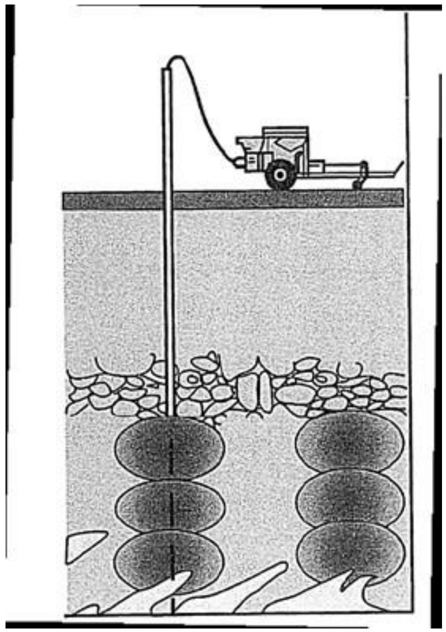


Dry Soil Mixing rig with mobile powder silo.

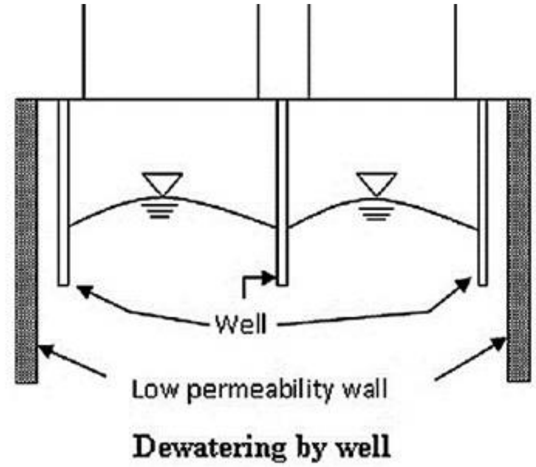


(A) Mixing tool is inserted to prepare the soil. (B) Lime and cement are injected and mixed as tool is rotated and raised. (C) Depths of completed columns are predetermined and controlled by on-board computer.

Soil mixing



Grouting



De-watering

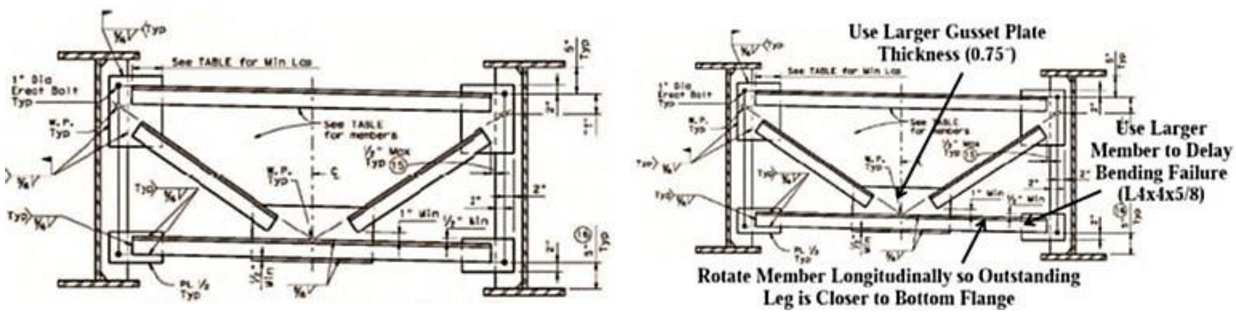
Figure 5-21. Retrofit of foundations (FHWA, 2011)

5.5.1 Strengthening measures for wind

Earthquake forces are high-intensity, short-duration (often seconds) phenomenon and as such, certain members are permitted to undergo demands that exceed their nominal capacity. By contrast, wind loading imposes long (sometimes days) loading on the bridge components. Wind gusts cause fluctuation in member stresses. For steel members, this is critical, and if the bridge connection details are not properly designed, they can experience failure due to fatigue. In general, to prevent fatigue failure, the stress in steel components needs to be kept to a fraction of the nominal capacity.

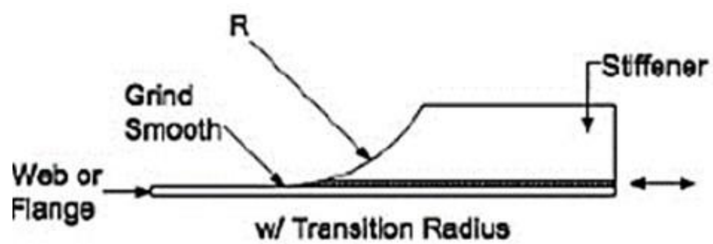
For steel girder bridges, a major source of fatigue cracking is at the connection of girders to the cross frames. The gap between the cross frames and girders causes secondary bending and concentrated stresses. Improved details (see Figure 5-22) can reduce stress in steel members and mitigate fatigue.

In general, small changes in the bridge connections can lead to enhanced performance. For example, for welded stiffener connections, increasing the transition radius from 50 mm to 150 mm or more will result in a four-fold increase (10/2.6) in the stress range that can be theoretically imposed on a connection for infinite number of cycles; see Figure 5-23.



Typical plans Improved detail for fatigue

Figure 5-22. Improved crossed frame connection (TXDOT, 2013)



Connection detail

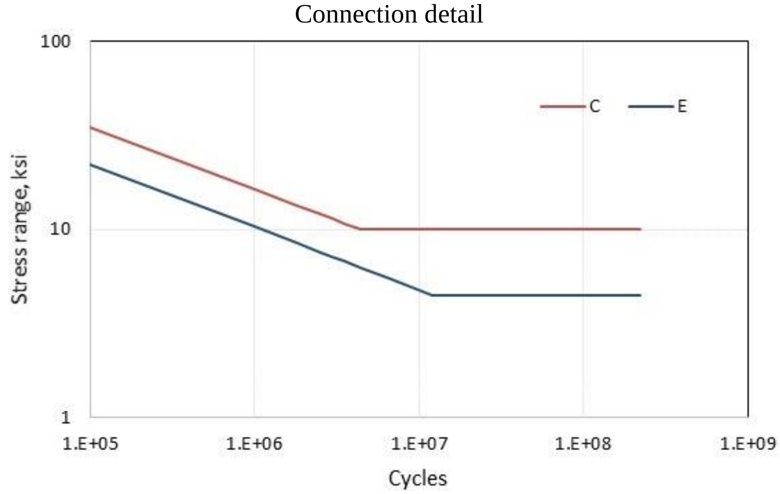


Figure 5-23. Fatigue performance for welded stiffeners (AASHTO, 2017)

For suspension bridges, wind studies are needed to determine the response of bridges to windstorms. Torsional stiffness can be increased by adding steel members to convert open sections to closed sections that have much higher torsional rigidity; see Figure 5-24. Other options include solutions to improve the flow of wind around the bridge superstructure. Examples include providing openings in fascia girders and solid sidewalks.

Original design Retrofitted bridge

Figure 5-24. Improved crossed frame connection (GGB, 2020)

5.5.2 Strengthening measures for flood

The most effective method for mitigation of flood damage is to provide protection to the bridge foundation. Assuming that the bridge superstructure is constructed high enough not to be inundated with floodwater, the flood hazard can be mitigated once the abutment and pier foundations are protected.

As a first step, the bridge foundation should be analyzed to ensure it could withstand the water pressure forces. If the foundation capacity is inadequate, then a solution similar to what is discussed in previous sections can be implemented. Once the abutment and pier foundation capacities have been confirmed, then riprap can be added to protect the ground around the foundation from washing out; see Figure 5-25. The size, gradation, volume, and placement of riprap should be confirmed based on hydraulics reports regarding the expected water floor at individual bridge sites.

Typical plans Retrofitted bridge

Figure 5-25. Application of Riprap (FHWA, 2017)

5.6 Paved roads (asphalt)

5.6.1 Overview

For roadways, earthquake or wind hazards are not considered unless the road is located on liquefiable soil or close to a fault experiencing permanent ground deformation (PGD). The mitigation for liquefaction is similar for paved roads and bridges. Flood presents the most critical hazard for paved roads.

5.6.2 Design codes

5.6.2.1 Seismic design in Dominica

As part of developing design requirements for coastal roads, University of the West Indies (UWI, 2001) undertook a training program in Dominica. Three key considerations were identified: i) safety, ii) environmental impact, and iii) cost (initial and life cycle). An example of a cross section of typical roads using flexible (asphalt) paving is presented in Figure 5-26.

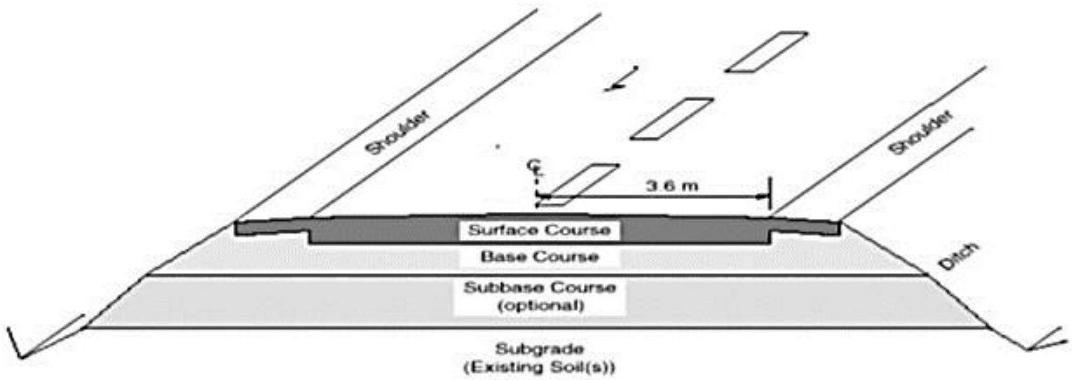


Figure 5-26. Cross section (UWI, 2001)

The document states that bases and subbases should have sufficient strength, uniformity, and consistency. The pavement thickness is set as a function of traffic loading and references AASHTO standards. It is noted that increasing pavement thickness by 150 mm will reduce the damage to pavement significantly.

5.6.2.2 Seismic design in Jamaica

In Jamaica, the National Work Agency (NWA) is responsible for developing design requirements for paved roads (2007).

Regarding flooding, the document states:

> *The drainage plan should correlate with the proposed ground level plan to minimize the passage of high storm flows on roadways and properties.*
>
> *All paved roadways are to be protected by kerb walls/ kerb and gutters, running parallel on each side of the road. Where design permits, well maintained grass verges are allowable in lieu of kerbwalls.*
>
> *Maximum recommended road grade shall be 15% for distances not exceeding 50 metres.*
>
> *No proposed road or lot should drain storm water directly unto an existing road or property without the specific permission from the National Works Agency*

Typical sections for arterial and major arterial roads are presented in Figure 5-27. Key requirements for paved road design are as follows:

- Base course is to be made of graded crushed aggregate. Grading varies from crushed stones of 0 to 37.5 mm with 10-15% gradation for various sieve sizes. Minimum California Bearing Ratio (CBR) of 80%
- Subgrade is to be made of graded crushed aggregate. Grading varies from crushed stones of 0 to 37.5 mm with 10-15% gradation for various sieve sizes. Minimum CBR of 30%
- Asphalt concrete (bitumen) should be based on a combination of coarse aggregate, fine aggregate, and mineral filler.
- The bitumen/binder is to be 5-7% by weight. Bitumen to fill 75-85% of voids.
- When minimum soaked CBR is less than 8%, treatment is required.

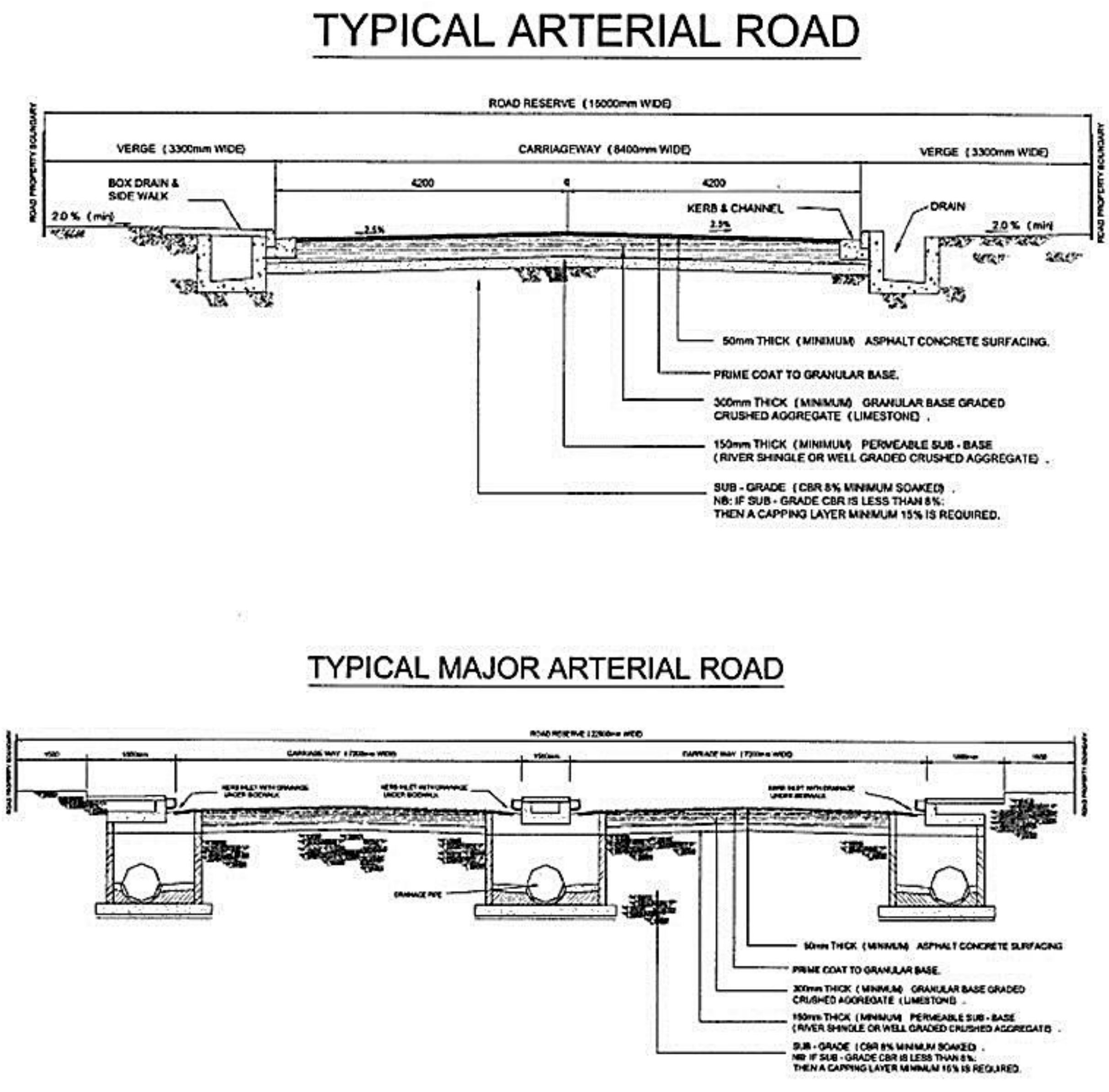


Figure 5-27. Typical road sections (NWA, 2007)

5.6.3 Fragility functions

For roadways, flood and the resulting landslide are the most critical natural hazards to consider. Several authors have developed flood fragility functions for roadways; see Figure 5-28. Note that although the functions differ somewhat, at inundation of 1.5 to 2 m, the damage ratio is 50% and increases to 100% with an immersion of 5 m.

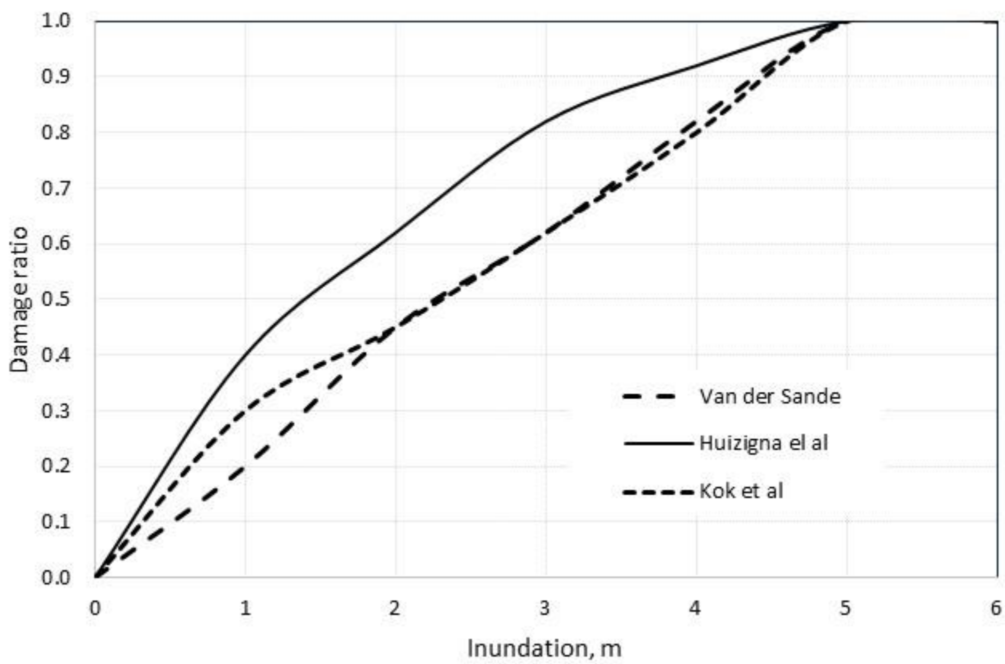


Figure 5-28. Flood fragility functions for roadways (various)

Flooding also results in additional vulnerability to earthquakes. Figure 5-29 (Argyrouddisi et al., 2018) presents the fragility function for highway embankment. Note that in the event of a 2-m inundation flood, a PGA of 0.4g (typical in the study countries) results in an increase of close to 100% in failure probability compared to the case where there is no flooding.

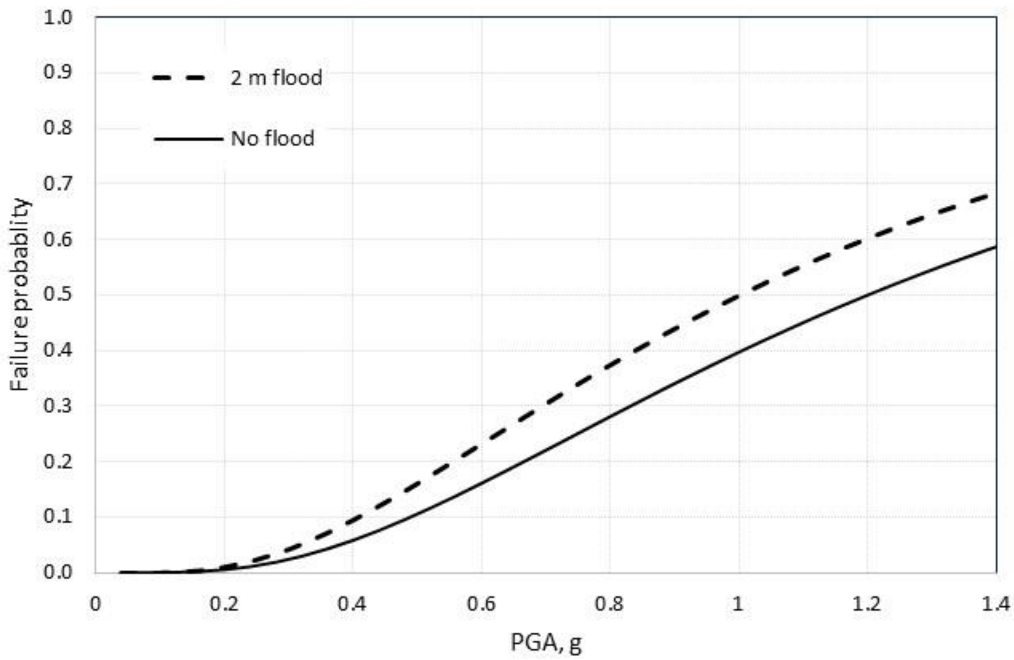


Figure 5-29. Roadway fragility functions for earthquake and flood

5.6.4 Strengthening measures

5.6.4.1 Compliance with AASHTO design procedure

For paved road design, AASHTO (2015) is used in the U.S. Key provisions for structural design of paved roads are summarized in this section. In general, pavements are classified into flexible, rigid or composite groups.

Flexible pavements are a mixture of asphalt or bituminous material placed on compacted aggregate. The design is based on traffic load being distributed along the depth of the cross-section and thus there is a grading in material quality with the highest quality (smaller cross-section area) being used near the top. This type of pavement is also referred to as Hot Mix Asphalt (HMA) pavement. Figure 5-30 presents a typical cross-section for HMA pavement.

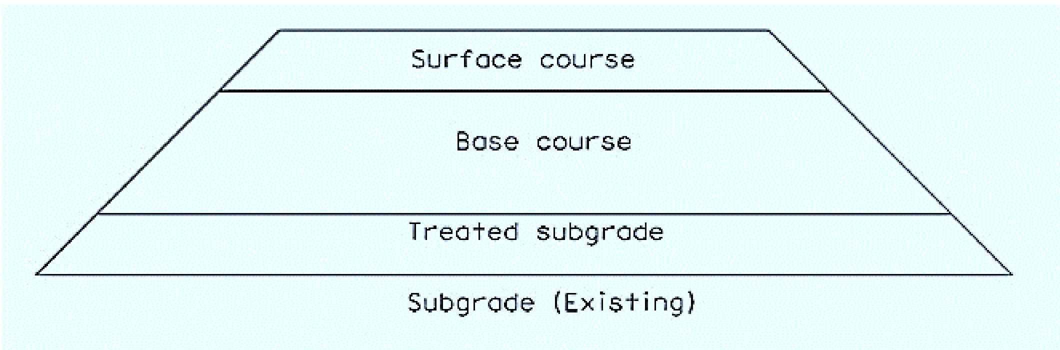


Figure 5-30. Typical cross-section composition for HMA pavement

HMA pavements are classified into three groups; see Table 5-10 and Figure 5-31 (AASHTO, 2015).

Layer	**Type**		
	Conventional	**Deep strength**	**Full depth**
Surface course	<150 mm HMA	Thick MHA	HMA

Optional	ATPB[10]	ATPB	ATPB
Base course	Unbound aggregate	Dense graded HMA over aggregate	HMA
Subgrade	Treated layer	Treated layer	Treated layer
Bottom	Foundation soil	Foundation soil	Foundation soil

Table 5-10. Types of HMA pavements (AASHTO, 2015)

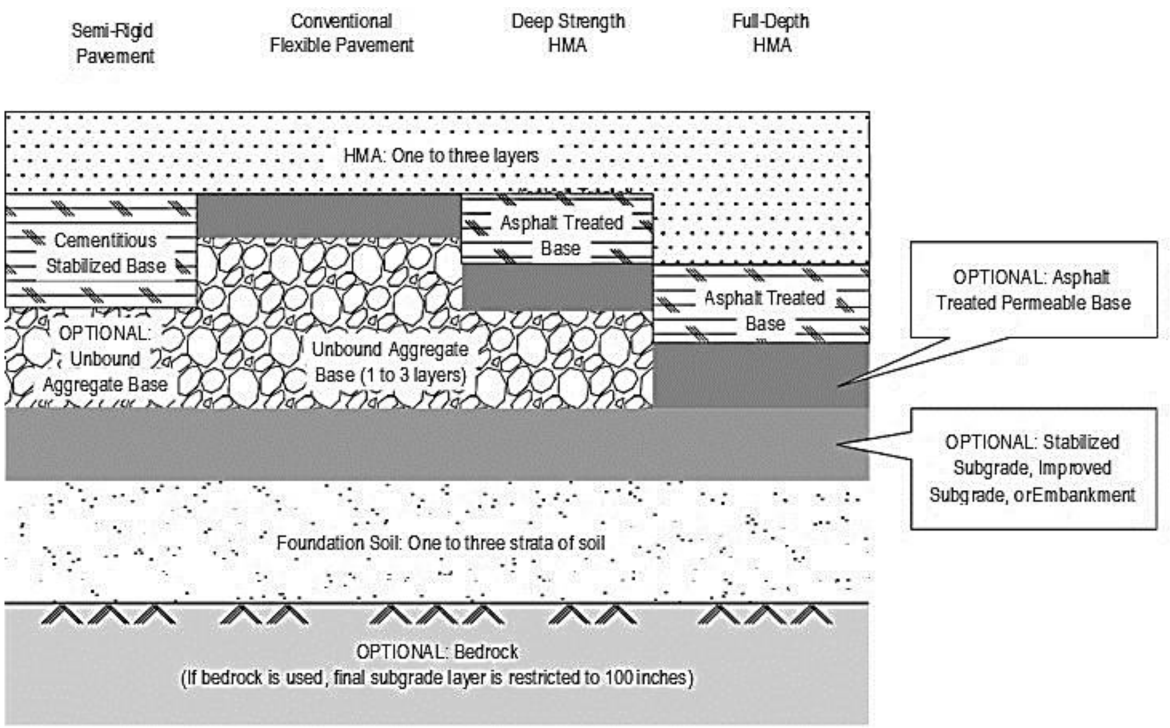


Figure 5-31. Cross-sectional view of HMA pavement types (AASHTO, 2015)

The design of pavements is based on accumulation of damage due to traffic and climate. Design performance criteria and reliability impact the cost and performance of the pavement. The recommended design criteria for primary roads is listed in Table 5-11 (AASHTO, 2015).

Performance criteria	**Threshold value at the end of design life**
Alligator cracking	20% lane area
Rut depth	12 mm
Transverse cracking length	132 m/km
Smoothness (IRI)	3100 mm/km

Table 5-11. Recommended design criteria, HMA pavement, primary roads (AASHTO, 2015)

Performance criteria is defined here as:

- Design life. The useful service life of a pavement that has deteriorated to a point where significant rehabilitation or replacement is required. In the U.S., a 50-year design life is typically used by state transportation agencies.
- Alligator cracking. Series of interconnected cracks as a result of wheel loading; quantified as percentage of lane area.
- Rut depth. Surface depression caused by permeant deformation in each pavement layer
- Transverse cracking (thermal). Normal to pavement centerline and caused by temperature variations

[10] Asphalt treated permeable base

- Smoothness. The parameter used to define pavement smoothness is International Roughness Index (IRI).

The moisture content of the pavement layer has a significant impact on the performance and durability of paved roadways. Flooding can submerge the roadway for days and damage subgrade. As such, it is important to ensure water can drain quickly from the roadways. Research on deleterious effects of flooding on roadways are presented in this section.

5.6.4.2 Strengthening methods for flood mitigation

The U.S. Army Corps of Engineers (USACE) has general responsibility for flood protection in the U.S. Currently, the Institute for Water Resources (IWR) is the organization working on development of a flood damage model for roadways (Davis et al., 2013). The key findings from their work are presented in this section. Flood damage to roadways has a number of mechanisms; see Table 5-12.

Damage mechanism	Mechanism
Overtopping	Flow normal to the centerline of roadway leading to erosion of embankment
Pumping	Removal of fine material from base or sub-base
Scour	Migration of channel into the toe of embankment
Drainage blockage	Loss of drainage capacity due to blockage or crushing
Embankment instability	Weakening of embankment caused by erosion
Wave action	Repeated overtopping due to breaking waves

Table 5-12. Flood damage consequences (Davis et al., 2013)

Overtopping is the most important damage mechanism and depends on velocity, duration, depth, and headwater and tailwater depths. Overtopping results both in loss of embankment and pavement and is directly related to soil type, duration, and overtopping depth; see Figure 5-32. For example, overtopping with a duration of 20 hours and overtopping of 1 m depth results in an approximate embankment pavement loss of 20% and 3%, respectively. As noted in the figure, the percent loss is significantly increased when inundation time is doubled.

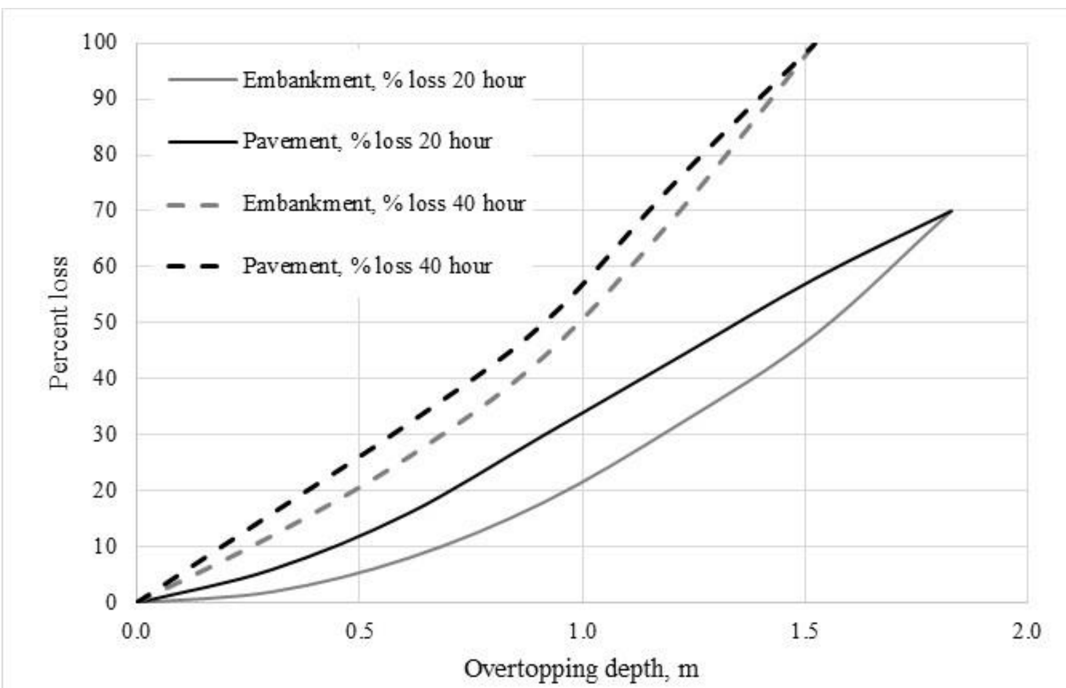
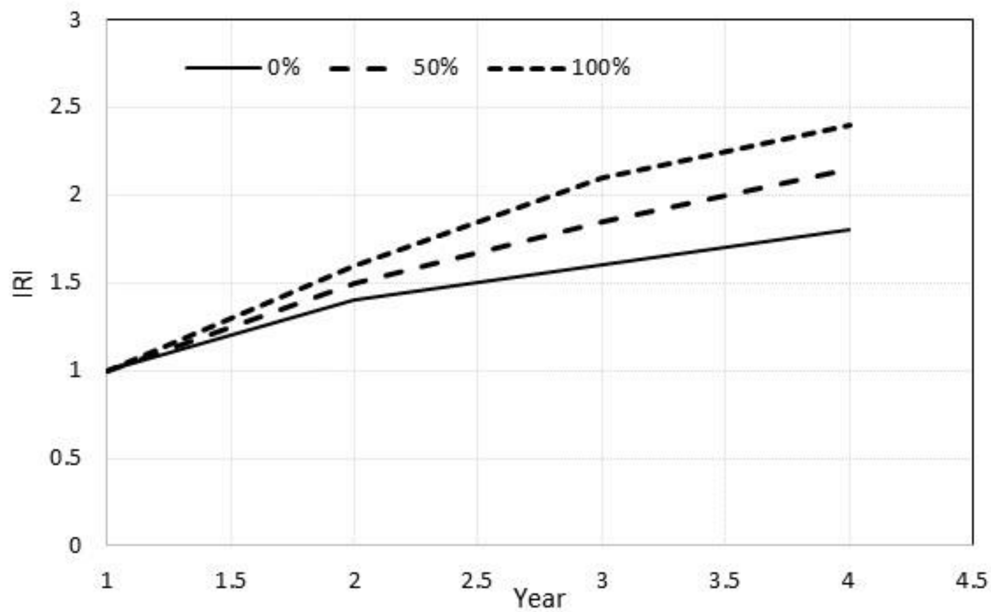


Figure 5-32. Pavement and embankment loss based on overtopping (adapted from Davis et al., 2013)

Khan et al. (2017) evaluated the performance of various types of pavements subjected to flooding in Australia. The key findings from their work are summarized in Figure 5-33. The following is noted:

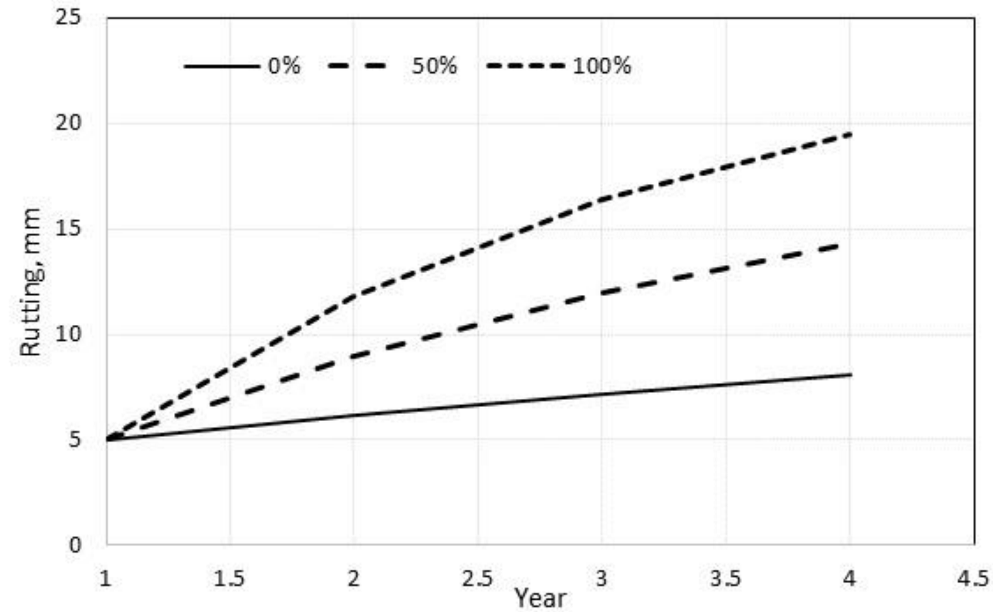
- For flexible pavements, IRI and rutting increase with an increased probability of flooding.
- Flexible pavements are more vulnerable to flood damage than rigid payments.
- Weak, flexible pavements are more vulnerable to flooding than strong flexible pavements.[11]

Pavement strength could be enhanced by adding sealants, increasing pavement depth, layer stabilization, and conversion to composite pavement, use of moisture-resistant materials, improved drainage, and reduced traffic loading during floods.

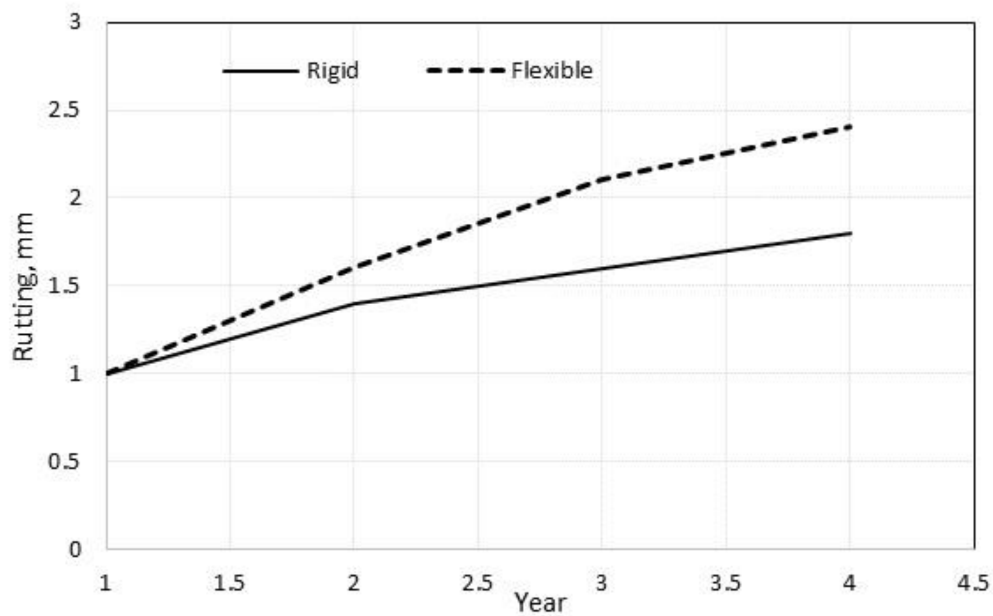


Pavement (IRI) damage, flood likelihood

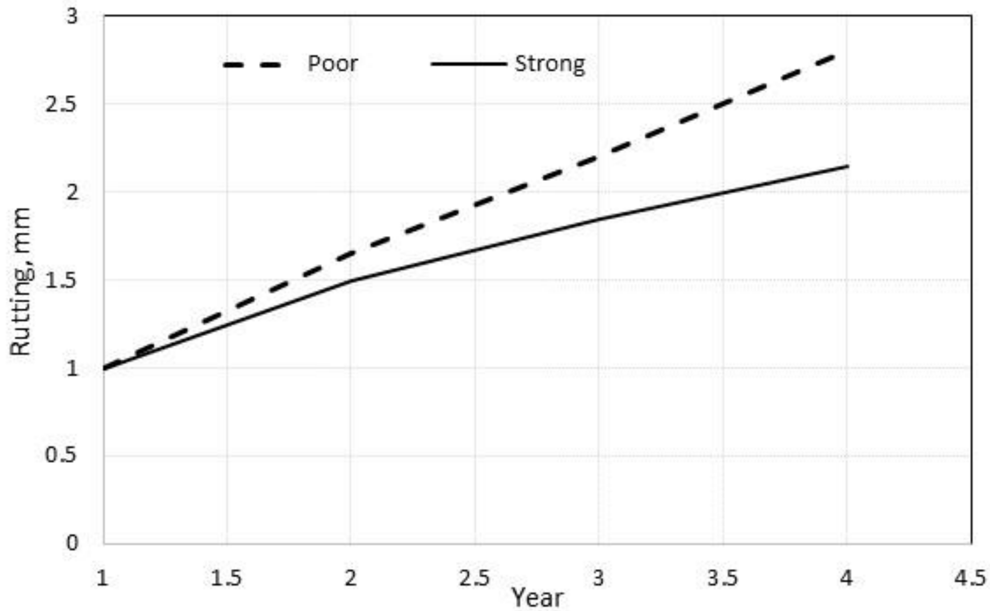
[11] Stronger pavements are the roads with thicker pavement depth



Pavement (Rutting) damage, flood likelihood



Pavement (Rutting) damage, pavement type



Pavement (Rutting) damage, pavement strength

Figure 5-33. Pavement damage as a function of probability of flooding (adapted from Khan et al., 2017)

5.7 Cost analysis

5.7.1 Introduction

Transport infrastructure managers need to balance programing for strengthening/retrofitting costs with other priorities, such as scheduled maintenance or replacement. One option to consider would be to incorporate strengthening incrementally as part of the maintenance program, and to identify the most vulnerable bridges and roads and prioritize them for early replacement. As discussed previously, the focus would be on existing bridges only, as it is assumed that new bridges, for example, have adequate freeboard (to prevent flooding), use more robust foundations (i.e., concrete or steel piles instead of timber ones), and have details with some ductility (to prevent collapse in earthquakes).

To implement the proposed strengthening techniques, it is critical to undertake a comprehensive plan that includes the following components:

- Perform a screening of all bridges and roads to supplement the existing inspection and maintenance records and detail vulnerabilities
- Bridges are more critical, as they are a larger investment, more difficult to repair after a major event, and could cause longer delays due to the longer detours required.
- Prioritize the most important and vulnerable bridges and road segment components, and program them for further evaluation and strengthening
- Develop a cost estimate for strengthening
- Perform a feasibility study

5.7.2 Assumptions in the cost analysis

In this section, some estimates of strengthening and replacement costs are presented. It is noted that most cost estimates are for the United States. However, it is assumed that the ratio of strengthening-to-reconstruction costs is similar in the U.S. and the Caribbean. The following is noted:

- Many countries in the Caribbean use U.S. design code for design of larger transport infrastructure.
- Many international companies use similar construction practices in the Caribbean as in the U.S.

- The income and cost-of-living expenses vary in subject countries and there is variation among the larger and smaller subject countries.
- For the most part, the labor cost is likely lower in the subject countries compared to the United States. However, many construction materials are likely more expensive, as they have to be imported.

Accordingly, for a typical transportation project, the percentage and total dollar amount of labor and material cost varies from country to country and compared to the U.S. This variation is likely reduced when large pools of transportation projects are considered and projects in a large number of subject countries are included in the cost analysis. More importantly, although it is likely that here will be a large variation in dollar value of cost and benefit computations (Δ = benefits associated with retrofitting, that is reduction in future repair costs, minus the capital cost of implementing the retrofit, that is the initial construction cost), in this chapter, the emphasis is on the unitless ratio of benefit-to-cost (i.e., benefit/cost). For this parameter, the difference in cost calculations among countries and between material and labor is less sensitive to variations than the total cost.

This last point is illustrated though a cost analysis example, shown in Table 5-13. A hypothetical case is considered here. Based on an assumed material-to-labor cost and retrofit-to-reconstruction cost ratio (biased towards material conservatively and because retrofitting typically is more labor than material intensive), the reconstruction and retrofit costs are computed for three cases: the U.S., a large subject country, and a small island country. As seen in the table, if the dollar value of benefit-cost (Δ) is used as a metric, the variation in computations can be off by a factor of 2 to 3 (160/77 or 77/26) in this example. However, when the ratio of benefit-to-cost (BCR) is considered, the variation is reduced to approximately 30% (variance between 3.8 and 2.8). Thus, it is expected that the benefit-to-cost ratio (BCR) for mitigation in the subject countries is not dissimilar to the U.S. case—and is used in this report in lieu of actual cost computations because of sparsity of local cost information—although the individual cost components and their contributions to the total cost can vary greatly. It is noted that similar analysis applies to other infrastructure considered in this report as well.

	Item	U.S.	Large Country	Small Island
Material-to-labor cost		0.5	3	4
Retrofit-to-reconstruction ratio	Labor	0.2	0.2	0.2
	Material	0.4	0.4	0.4
Reconstruction	Labor cost	70	10	50
	Material cost	35	30	200
	Total cost	105	40	250
Retrofit	Labor cost	14	2	10
	Material cost	14	12	80
	Total cost	28	14	90
Financial analysis	Δ	77	26	160
	BCR	3.8	2.9	2.8

Table 5-13. Hypothetical cost and benefit calculations

For transport infrastructure, there are three cost types associated with damage as a result of natural hazards:

- Structural or primary cost. This is the cost associated with structural damage and the requirements to reconstruct or repair the damage that was caused by the event.
- User or secondary cost. This is the cost that results from longer commute times, traffic delays, use of detours, wear and tear on vehicles, loss of revenues for managing agencies, and less safety for traveling in the construction zone.

- Tertiary costs. This is the cost corresponding to adverse socioeconomic impacts and cascading effects of damage to transport infrastructure. Since transport infrastructure provides connectivity and allows for movement of goods and people, a disruption to service could economically impact a community. Furthermore, interruption to transport infrastructure will delay recovery and negatively impact other infrastructure.

In this report, the cost-benefit calculations are conservatively based on the structural component only and show the cost-effectiveness of mitigation. Once indirect costs are factored in, the BCR can significantly increase and make mitigation even more appealing to decision makers.

Finally, in the aftermath of a major natural disaster that causes damage to the built environment, there is usually high demand for skilled labor and scarcity of construction material. This is due to the high demand immediately after natural hazards. As such, there is a cost premium associated with repair and reconstruction work. This inflation—likely more pronounced in smaller island countries, which depend heavily on importation of construction material—will further tilt the balance in favor of mitigation and make the benefits even more cost effective. This additional cost premium associated with savings as a result of mitigation—applicable to transport, water, power and critical building infrastructure as well—was not included in this report and therefore, the computations are conservative as they underestimate the benefits of mitigation.

In this section, some estimates of strengthening and replacement costs are presented. It is noted that most cost estimates are for the United States. However, it is anticipated that the ratio of strengthening-to-reconstruction costs are similar in the U.S. and the Caribbean, because many countries in the Caribbean use U.S. design code for design of transport infrastructure, and many international companies use similar construction practices in the Caribbean as in the U.S.

5.7.3 Reconstruction costs

5.7.3.1 Cost basis

The construction cost for bridge and road construction in the subject countries is presented in Table 5-14.

Country	Description	Cost, US$ million/km
Bahamas	82x7 m steel truss bridge and road improvement, US$6 M	
Barbados	14.8 km highway and bridges, US$9.4 million 31 km road, US$50 million	1.6
Belize		1.0
Dominican Republic	106 km highway, US$150 million	1.0
Guyana		1.2
Haiti	83 km highway, US$238 million	3
Jamaica	67 km 4-lane highway, US$600 million	9
Antigua and Barbuda	8.7 km highway, US$17.5 million	2
Saint Lucia		1.5
Median		1.5

Table 5-14. Transport infrastructure construction in subject countries (various sources)

5.7.3.2 Bridges

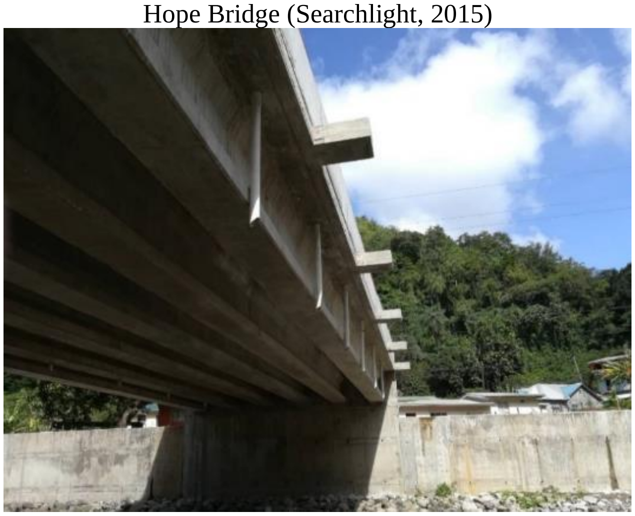
In Florida, the Florida Department of Transportation (FDOT) maintains a database for construction cost of new bridges (2020). These costs can be used to estimate the replacement (reconstruction) cost of bridges in the Caribbean. Example data for the type of bridges common in the Caribbean, based on the data gathered by the FDOT, are presented in Table 5-15.

Type	Span	Cost US$/m^2 of deck
RC slab simple span	Short	$1,250-1,750
Precast concrete simple span		$1,200-2,150
Concrete deck, steel girder, simple spans	Medium or long	$1,350-$1550
Concrete deck, steel girder, continuous spans		$1,550-$1,800
Concrete deck, precast girder, simple spans		$1,000-$1,550
Concrete deck, precast girder, continuous spans		$1,050-$2,250
Bridge removal	--	$400-$650

Table 5-15. Bridge construction cost (adapted from FDOT, 2020)

In the aftermath of hurricanes and floods in 2013, a number of bridges in Saint Vincent and the Grenadines were damaged and required replacement. Examples are presented in Figure 5-34. Key information for these bridges is presented in Table 5-16 (Searchlight 2015, 2018). Note the significant cost associated with reconstruction of damaged bridges in Table 5-16. As discussed later, it is more cost effective to implement retrofitting and mitigation measures prior to natural disasters.

Hope Bridge (Searchlight, 2015)



Cumberland Bridge (Alvair, 2015)

Figure 5-34. Replacement bridges after flood damage in Saint Vincent and the Grenadines

Activity	Type	Size	Cost US$
Hope Bridge replacement and 3 other bridges, plus river embankment	PC girder	24 * 8 m	4.2 million
Cumberland Bridge replacement, repair for 3 minor bridges, embankment protection	PC girder	26 m	5.8 million

Table 5-16. Replacement bridge information for Saint Vincent and the Grenadines (Searchlight 2015, 2018)

Table 5-17 presents examples of unit costs for new bridge construction for four bridges that were constructed in Saint Vincent and the Grenadines in 2020. It is noted that the unit costs are comparable with U.S. practice of Table 5-15, it is reasonable to use the U.S. cost data as a baseline for the subject countries when comparing costs associated with reconstruction and retrofitting.

Bridge	**Type**	**Length, m**	**Cost US$/m^2 of deck**
1	RC girder	8	$2,100
2	RC girder	19.29	$1,000
3	RC girder	8	$2,100
4	RC girder	15.8	$1,050

Table 5-17. 2020 Bridge construction cost of four new bridges in Saint Vincent and the Grenadines

As part of the Trinidad and Tobago Roads and Bridges Programme of the National Highway Programme under the Inter-American Development Bank (IDB), 40 bridges were reconstructed; see Table 5-18 (Khan-Kernahan, 2013).

Bridge type	**Spans**	**Span, m**	**No.**
Single cell box girder	1 1	6 m 9 m	14 12
Two cell box girder	2	6 m	1
Three cell box girder	3	9 m	2
Four cell box girder	4	6 m	1
Slab bridge	1	9 m	1
Precast girder	1	25 m	8

Table 5-18. Bridge construction cost in Trinidad and Tobago (adapted from Khan-Kernahan, 2013)

As part of the USAID-Organization of American States (OAS) Caribbean Disaster Mitigation Project (CDMP), a bridge in Saint Lucia was retrofitted and another one was reconstructed. CDMP (1998) writes:

> *a) The cost of reconstruction work required to stabilize the banks of the river at this bridge site is estimated to be US $120,000.*
>
> *b) This cost when deflated to 1975, the year of construction of the present bridge, is 17.4% of the 1975 construction cost.*
>
> *c) The estimated costs of additional engineering and construction that would have been required in 1975 to prevent the damage which occurred in 1996 is US $20,000 or 10.8% of the original construction cost.*
>
> *d) The designers of the project in 1974 had made provisions for protection of the riverbanks. The immediate cause of the damage from the heavy rainfall in 1996 was the due to the reduction of the waterway in 1995. If this construction had not been undertaken, there may have been less damage to the north bridge approach. The riverbank protection now recommended by the Ministry of Communications and Works would have prevented the damage.*
>
> *The Caico bridge that was damaged by floodwaters was constructed in 1995. It replaced a bridge that was damaged by the 1994 flood. The superstructure was a Bailey bridge supplied by the British Development Division (BDD) and installed by the Ministry of Communications and Works of St. Lucia. The bridge was a single span of about 50 feet and the superstructure rests on existing stone abutments. This bridge was examined in some detail, as the original bridge constructed more than 20 years ago was destroyed by the 1994 floods. The area served by this bridge is an important banana producing area.*

The chief engineer reported that adequate wing walls and riverbank protection were not constructed for the reconstructed bridge due to lack of funding.

In Table 5-19 (CDMP, 1998) and Table 5-20 (adapted from Dannion Engineering, 2020), note the cost effectiveness of mitigation for the Troumassee and Caico bridges. Figure 5-35 presents photographs of the bridges.

Bridge	Type	Span	Activity	Cost, US$	Reconstruction-to-mitigation ratio (BCR)
Troumassee bridge	Steel truss	2 x 21 m	Original (1975) construction cost	185,000	32,000/20,000=1.6
			Mitigation not incorporated in 1965	20,000 (11%)	
			Reconstruction: add concrete-filled steel piles and raise pile cap in 1996	120,000	
			Reconstruction cost (deflated to 1975)	32,000 (18%)	
Caico bridge	Bailey Bridge	24 m	Reconstruction (1997)	317,000	>2[12]

Table 5-19. Construction cost of bridges in Saint Lucia (adapted from CDMP, 1998)

Bridge	Type	Span	Description	Cost, US$
Bonne Terre Bridge	Concrete	12 m	19 m wide, 4 lanes, 120-year service life, designed for 50-year flood	5,600,000

Table 5-20. Construction cost of bridges in Saint Lucia (adapted from Dannion Engineering, 2020)

Caico Bridge

Bonne Terre Bridge

Figure 5-35. Saint Lucia bridge construction (Dannion Engineering, 2020)

5.7.3.3 Paved roads

In Florida, the Florida Department of Transportation (FDOT) maintains a database of construction costs for new paved roads (2020). These costs can be used to estimate the replacement (reconstruction) cost of roads in the Caribbean. Example data for the type of paved roads common in the Caribbean, based on the data gathered by the FDOT, are presented in Table 5-21. Note that the costs are the same order as those of the subject countries listed in Table 5-14.

[12] The bridge was reconstructed after the 1994 storms. The reconstructed bridge was destroyed in the 1996 storm and had to be reconstructed again.

Location	Type	Cost US$/km
Rural	Two-lane undivided; new	1,600,000
	Four-lane divided; new	3,000,000
	Six-lane highway, new	4,600,000
	Two-lane undivided; re-surface	300,000
	Four-lane divided; re-surface	800,000
	Six-lane highway; re-surface	1,000,000
Urban	Two-lane undivided; new	3,500,000
	Four-lane divided; new	8,100,000
	Six-lane highway, new	8,500,000
	Two-lane undivided; re-surface	350,000
	Four-lane divided; re-surface	800,000
	Six-lane highway; re-surface	1,200,000

Table 5-21. Bridge construction cost (adapted from FDOT, 2020)

In Jamaica, more than 90% of roads use asphalt pavement. Examples of cost data for pavement are presented in Table 5-22.

Pavement type	Design life	Cost US$/m^2 of road
Asphalt (flexible), 40 mm thick	15 years≤	2,000
Concrete (rigid), 200 mm thick	20-50 years	8,000

Table 5-22. Example of pavement costs in Jamaica (adapted from Jamaica Gleaner, 2018)

Examples of cost data for various pavement types are presented in Table 5-23. The data are based on aggregate results of 3,300 data points from a number of countries, including Belize (3 data points), the Dominican Republic (43 data points), Haiti (8), and Jamaica (1). Note that concrete pavements have a higher initial cost but given that this pavement has two-to-three times the design life of asphalt pavement, it could be cost-effective. It is also noted that the reconstruction cost is assumed to be of the same order as original construction of roadways.

Construction	Cost per km
New 4-lane highway	US$2,200,000
New 2-lane highway	US$750,000
New 1-lane road	US$92,000
Widening and reconstruction	US$850,000
Concrete pavement	US$540,000

Table 5-23. Example of pavement component costs (adapted from WBG, 2018)

Table 5-24 presents examples of unit costs for new paved road construction for three roads constructed in Saint Vincent and the Grenadines in 2020. It is noted that the unit costs are comparable with US practice and the World Bank data presented above and as such, it is reasonable to use the U.S. cost data as a baseline for the subject countries when comparing costs associated with reconstruction and retrofitting.

Road	Type	Length, km	Cost per km
1	Asphalt	9.5	US$160,000
2	Concrete	3.6	US$1,800,000
3	Concrete	1.5	US$1,600,000

Table 5-24. Paved road construction cost in Saint Vincent and the Grenadines

5.7.4 Strengthening costs

5.7.4.1 Bridges

Chandrasekaran and Banerjee (2014) examined the cost effectiveness of seismic retrofitting of typical concrete bridges in California. The bridge examined was 240 m long, 13 m wide, and comprised of five spans with a maximum span of 53 m. They considered two hazard cases: i) earthquake with PGA of 0.8 g and ii) flood causing 2.4 m of scour. Note that these hazard values could also correspond to the expected values in the Caribbean.

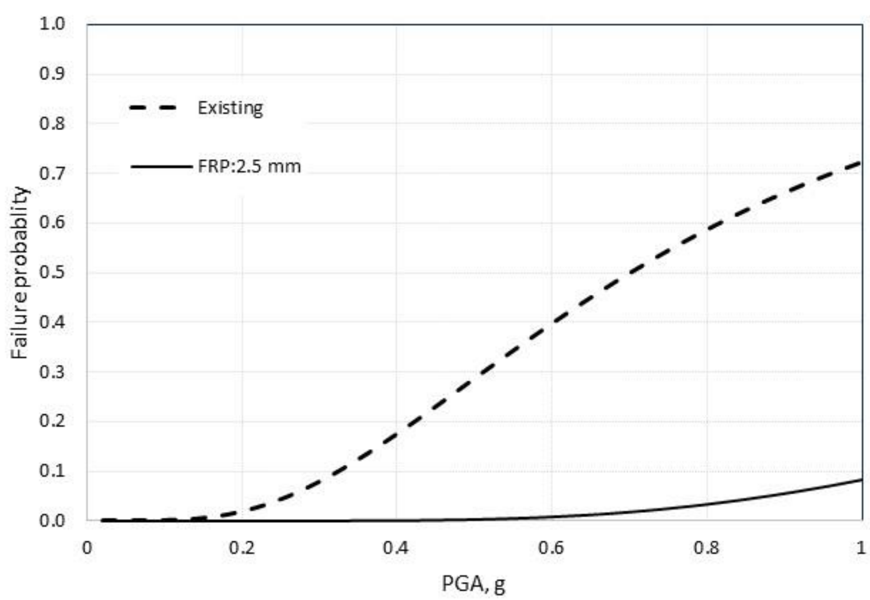
They examined several cases (see Table 5-25) and they performed numerical analyses using a suite of earthquake and flood events.

Case	Fiber-reinforced polymer (FRP) thickness for column retrofit
0 (as-is)	0.5
1	1.0
2	1.5
3	2.0
4	2.5

Table 5-25. Bridge column retrofit scenarios (adapted from Chandrasekaran and Banerjee, 2014)

Examining the cases of no mitigation and a mitigation with 2.5 mm of FRP for concrete columns, the following is noted when FRP retrofitting is implemented; see Figure 5-36.

- Analysis was conducted for earthquake after an earlier flood event caused 2.4 m of scour
- Probability of failure (at PGA 0.8g) reduced from 60% to 5%
- Loss ratio reduced from 95% to 60%
- Resiliency increased from 58% to 72%



Probability of failure

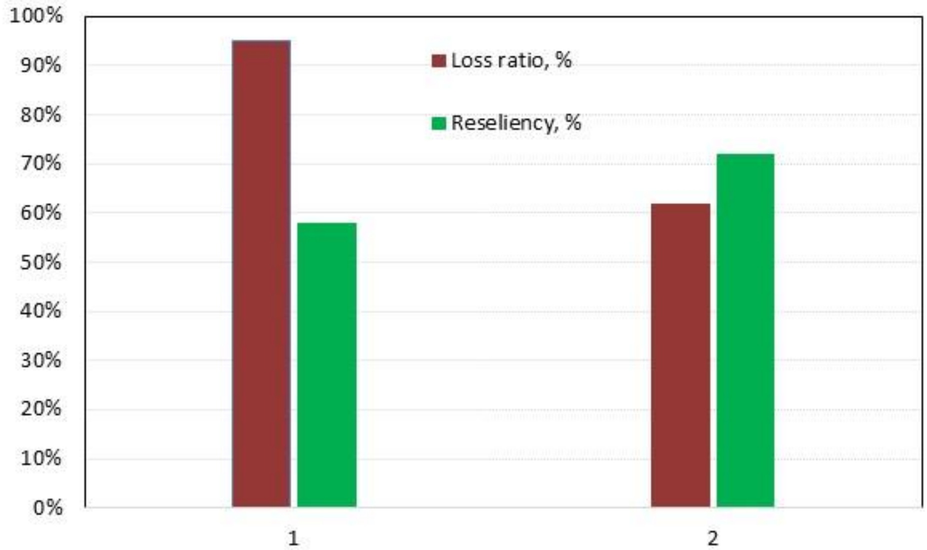


Figure 5-36. Retrofit effectiveness of bridge columns (adapted from Chandrasekaran and Banerjee, 2014)

Note the following:

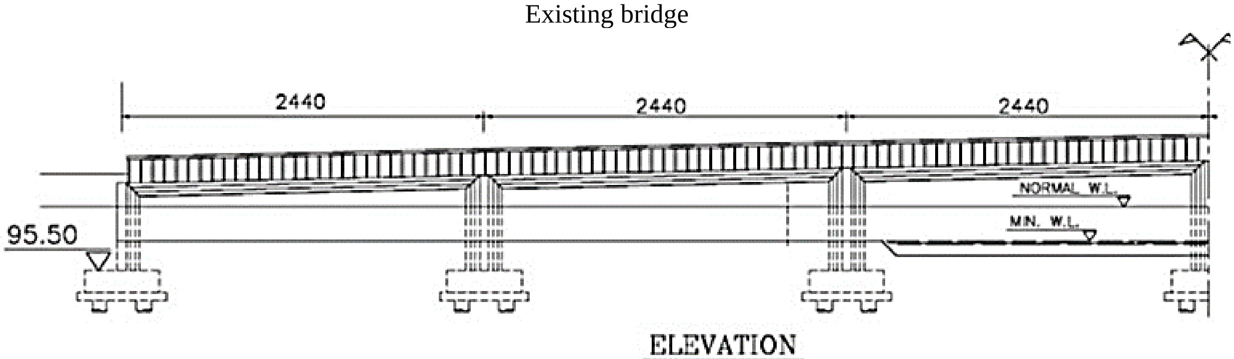
- The retrofit cost was estimated by the authors as US$900/m^2 of deck
- For concrete bridges with a 53-m span, the cost of new/reconstruction is approximately US$3,500/m^2 of deck (Caltrans, 2020)
- Flood and seismic retrofitting cost was approximately 25% of new construction and resulted in a significant decrease in failure and loss.

Asadi et al. (2020) examined the cost and benefits associated with retrofit of an existing bridge in a developing country. The bridge, constructed in 1976, has six spans of 24.4 m long. Each pier has three piers. The bridge was not designed for seismic loading; see Figure 5-37. The bridge piers are brittle and require retrofitting. Additionally, the bridge pier reinforcement is subject to corrosion and flood hazard can cause pier scour at the site. The site has moderate seismicity, with PGA of 0.35g. The scour depth was estimated based on a number of scenarios and a depth of 2 m was computed. The authors examined the pier retrofit using FRP and adding reinforcement to deficient piers. Note that this bridge, the site hazards, and the proposed retrofit could be typical of a case in the subject countries in this report.

The authors noted the following:

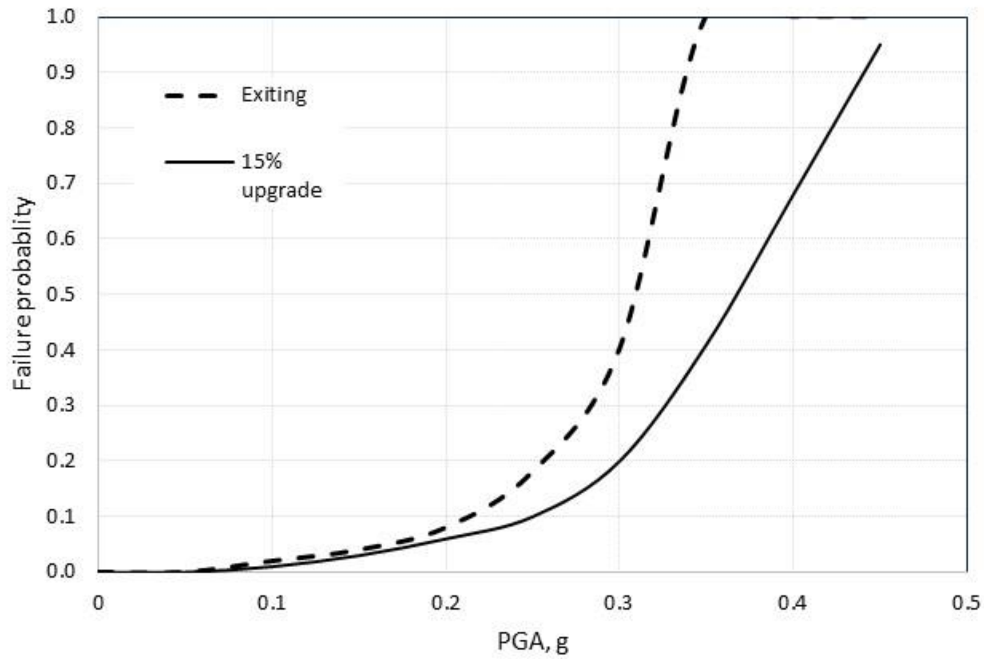
- In the event of an earthquake with PGA of 0.45 g, the existing bridge will likely collapse, whereas the retrofitted bridge will experience moderate and repairable damage.
- Compared to the existing bridge, the retrofitted bridge has a higher capacity and lower loss ratio; see Figure 5-38.
- The retrofit is cost effective and has a benefit-to-cost ratio of greater than 1.0.

Existing bridge

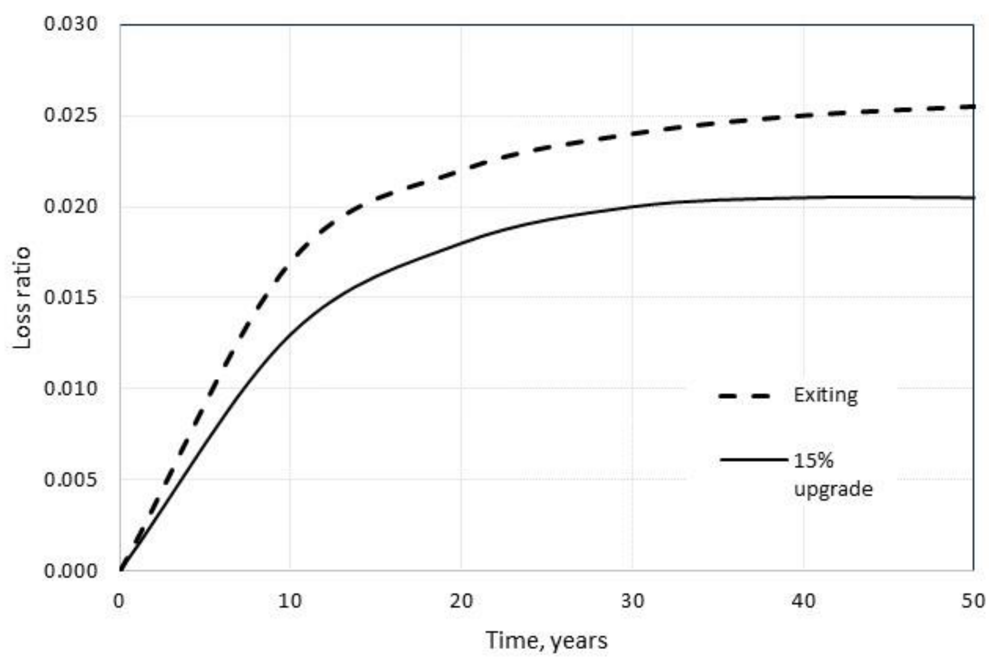


Elevation

Figure 5-37. Bridge under investigation (Asadi et al., 2020)



Failure probablity reduction



Loss reduction

Figure 5-38. Loss reduction due to retrofitting (adapted from Asadi et al., 2020)

Lee et al. (2004) examined concrete girder bridges. Note that this is one of the most common bridge types in the Caribbean and as discussed earlier, has column, bearing, and unseating vulnerabilities. The methodology took into account both vulnerability and future failure costs. They examined steel jacket retrofitting and noted that:

> *Based on the simulated results for existing and retrofitted bridges, the application of steel jackets is found to be a positive solution to reduce the seismic damage risk of bridges.*

The U.S. Department of Transportation (USDOT) provides the methodology for determining merits of retrofit and reconstruction (2016). In their report, the authors examined a steel girder bridge, which could be similar to a bridge or flyover in the Caribbean. The 5-lane bridge has a 30 degree skew, has three spans, and is 91-m long and 32-m wide; see Figure 5-39. As part of retrofit, demotion and reconstruction of one abutment was considered. The work had a 233-day schedule. The results of the simulation are presented in Table 5-26. They considered two activities, listed below, with the associated costs presented in Table 5-26.

- Partial: Partial depth removal of the backwall, deck, and total dam with Class A concrete
- Full: Full depth removal of the dam with Class A concrete

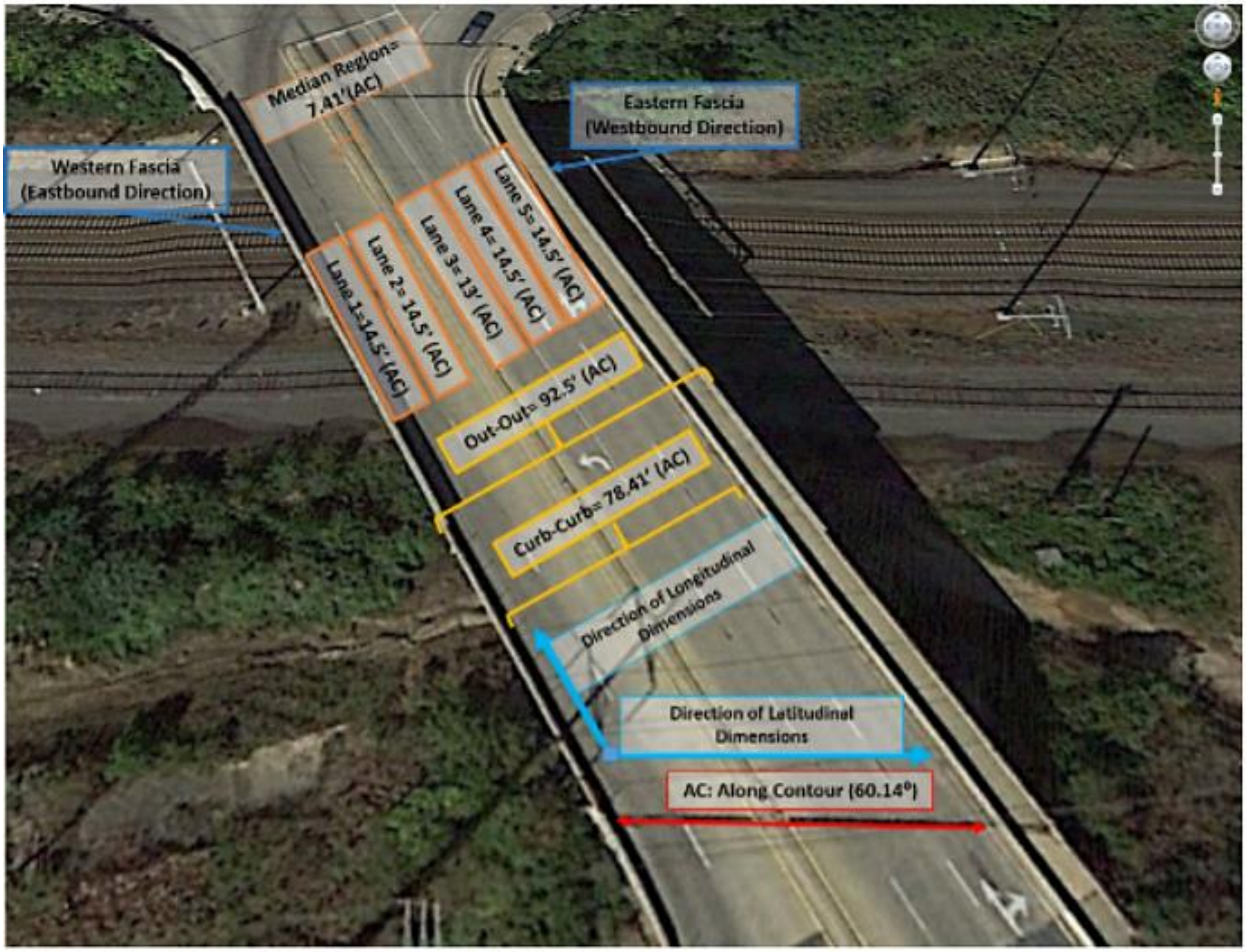
Activity	**Initial cost US$**			
	Owner	**User**	**Other**	**Total**
Partial	$2,600	$19,000	$600	$22,200
Full	$21,200	$87,000	$2,800	$111,000

Table 5-26. Example of cost computations (USDOT, 2016)

USDOT (2016) looked at various scenarios; see Table 5-27. Note that early mitigation is more cost effective than a delayed full reconstruction.

Scenario	**Year 1**	**Year 12**	**Year 14**	**Overall cost**
I	Full	Partial	--	US$175,200
II	Full	--	Full	US$222,000

Table 5-27. Example of scenarios (USDOT, 2016)



Photograph of bridge

Figure 5-39. Case study bridge (adapted from USDOT, 2016)

5.7.4.2 Paved roads

Examples of cost data for various pavement types are presented in Table 5-28. As discussed earlier, the data are based on aggregate results of 3,300 data points from a number of countries, including Belize (3 data points), the Dominican Republic (43 data points), Haiti (8), and Jamaica (1). Note that the retrofitting costs are significantly less than the reconstruction costs.

Construction	Cost per km
Upgrading	US$250,000
Asphalt resurfacing	US$65,000
Bituminous Pavement Preventive Treatment	US$7,000
Strengthening	US$140,000
Surface treatment	US$25,000
Preventive maintenance	US$5,000
Routine maintenance	US$2,000

Table 5-28. Example of pavement rehabilitation costs (adapted from WBG, 2018)

5.7.5 Life-cycle cost

For transport infrastructure, life-cycle cost analysis (LCCA) is typically used in design. This topic will be further discussed in Chapter 8.

5.8 Discussion

Throughout the Caribbean, there is extensive transport infrastructure, including many bridges and thousands of kilometers of paved roads. The countries are also located in one of the most hazard-prone areas in the world, experiencing a large number of hurricanes and floods annually and earthquakes periodically. These natural hazards have caused significant damage to bridges and roads in recent years, causing economic distress, delayed recovery, impacts to travelling and public life, and interruptions of the movement of goods and services. Accordingly, it is critical to plan and implement measures that result in resilient transport infrastructure. For bridges, there are a number of components that are the most vulnerable to earthquakes and flooding. For these components, effective retrofitting and mitigation methods are available that reduce the expected damage to infrastructure and are cost effective. Table 5-29 presents a summary of the findings related to the most critical components of transport infrastructure in the subject countries.

Subsector	**Hazard**	**Vulnerable Component**	**Resilience Measure**	**Cost, % of initial cost (Est.)**	
				Improvement Cost	**Vulnerability reduction**
Paved roads	Flood	Paved surface	Add sealant	2%	20%
			Strengthen (thicker pavement)	20%	50%
			Concrete surface	40%	80%
Bridges	Earthquake	Columns	Add FRP or steel casing	10%	60%
		Bearing	Add cable restrainers	2%	50%
		Abutment, pier seats	Add concrete bolsters	5%	50%
		Steel cross frames	Fatigue-resistant details	10%	80%
	Flood	Foundations, abutments	Add riprap	5%	20%
	Flood, liquefaction		Add new piles	40%	80%

Table 5-29. Summary of findings for transport infrastructure in the Caribbean

6. POWER INFRASTRUCTURE

6.1 Introduction

Improvement in resiliency of the power infrastructure (see Figure 6-1) has a two-fold beneficial impact for a country: i) by reducing physical damage to the infrastructure components, economic losses are reduced; and ii) the continuous operation of power substructures allows other infrastructure, such as hospitals and emergency centers, to continue to function, which allows expedited overall community recovery. Substations and transmission lines are in particular critical infrastructure lifelines and will be emphasized in this chapter. For power plant structures, many of the measures discussed for building infrastructure apply.

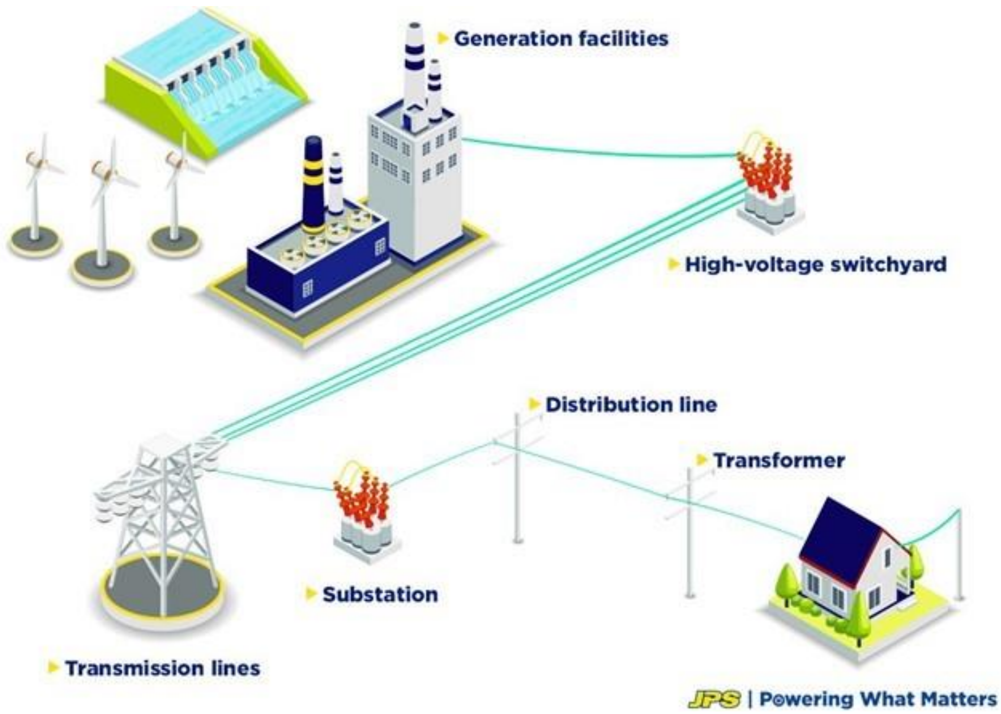
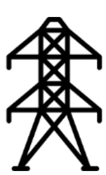


Figure 6-1. Power infrastructure

For power infrastructure in the Caribbean, the hazard and infrastructure components listed in Table 6-1 will be used. As noted in the table, for power plants and substations, earthquakes, and to a lesser extent, flooding, are critical, whereas for distribution systems, wind is the most critical hazard. It is assumed the discussion presented herein applies to all countries in the Caribbean, as power sector construction is likely similar across the subject countries due to significant cooperation between utility providers.

Sector	Subsector	Type	Components	Hazard
 POWER	Power plants	Low-rise RC	Design	Earthquake
			Grade construction	Flood
			Nonstructural components	Earthquake
			Foundation	Earthquake, Liquefaction
	Substations	Control buildings	Design construction	Earthquake
			Grade construction	Flood
			Battery racks, cable trays, etc.	Earthquake
			Foundation	Earthquake, Liquefaction
		Transformers	Anchorage	Earthquake

		Frames	Steel members	Earthquake, wind
		Equipment	High-voltage units	Earthquake
	Transmission lines	Lattice towers	Tower	Wind, Earthquake
			Foundation	Flood, Liquefaction
			Lines	Wind

Table 6-1. Analysis matrix for power infrastructure

In this report, the main transmission system (lines and poles) transferring power from generation centers to switchyards and substations will be discussed. The distribution systems carrying electricity to individual structures are not considered. Figure 6-2 shows the distinction between transmission and distribution lines.

Transmission towers and lines

Distribution poles and lines

Figure 6-2. Transmission and distribution systems (various)

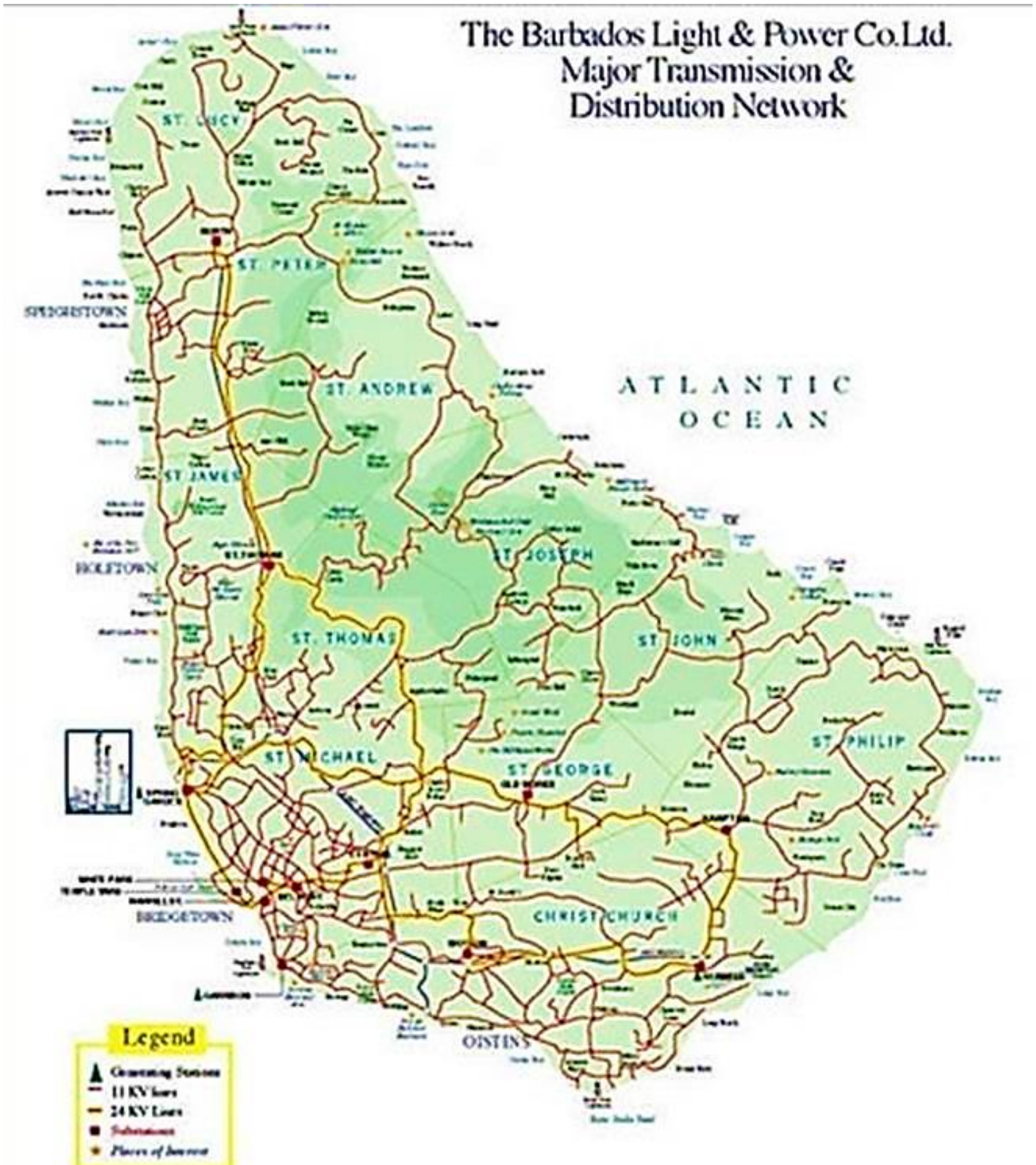
6.2 Geographical distribution

In the subject countries, the following utility companies are responsible for generation, transmission, and distribution of electrical power; see Table 6-2.

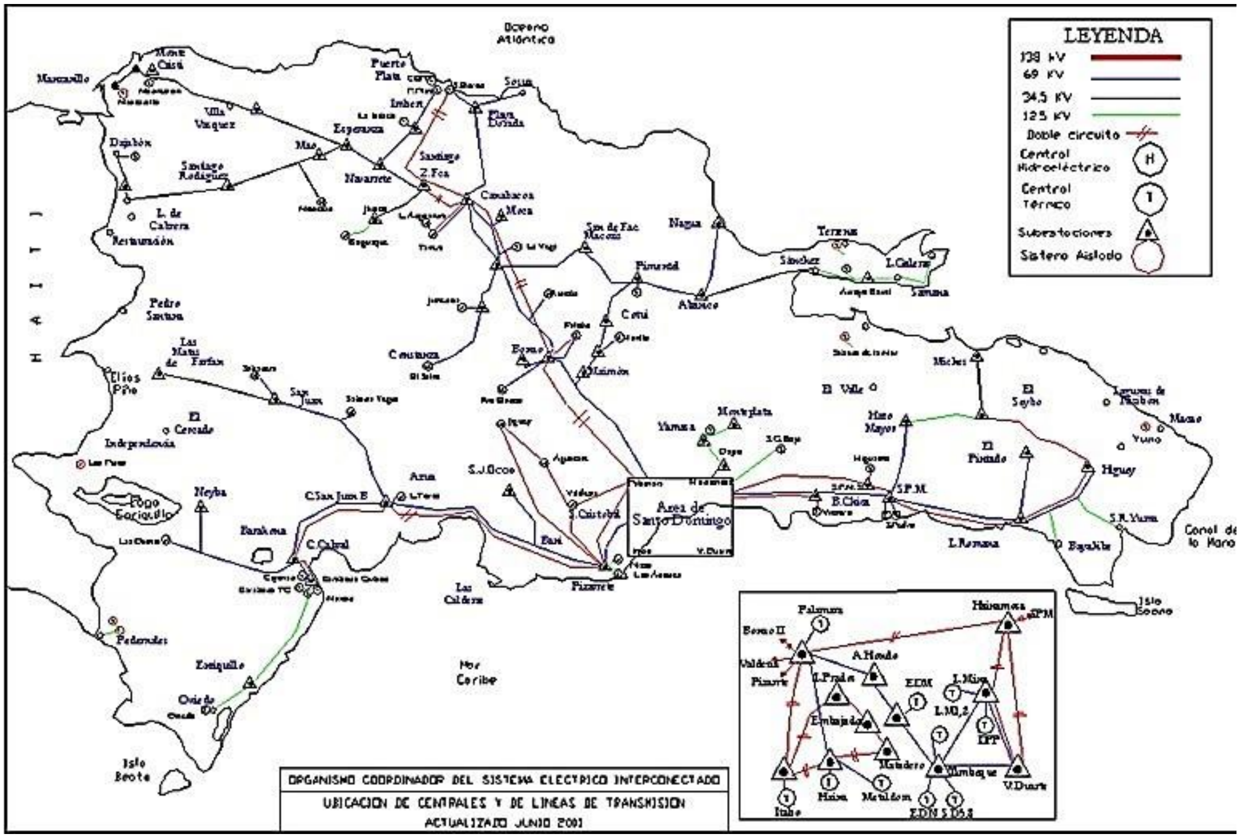
Country	Utility
Suriname	N.V. Energie Bedrijven Suriname (EBS)
Trinidad and Tobago	Trinidad and Tobago Electricity Commission (T&TEC)
Guyana	Guyana Power Light (GPL)
Belize	Belize Electricity Limited (BEL)
Haiti	Electricité d'Haïti (EDH)
Dominican Republic	Dominican Corporation of State Electricity Companies (Corporación Dominicana de Empresas Eléctricas Estatales - CDEEE)
Antigua and Barbuda	Antigua Public Utilities Authority (APUA)
Dominica	Dominica Electricity Services (DOMLEC)
Grenada	Grenada Electricity Services Ltd. (Grenlec)
Saint Kitts and Nevis	St. Kitts Electricity Company Ltd. (SKELEC)
Saint Lucia	Light and Power Holdings (LUCELEC)
Saint Vincent and the Grenadines	St. Vincent Electricity Services Limited (Vinlec)
Sint Maarten	Electricite de France (EDF)
Barbados	Barbados Light & Power Company Limited (BL&P Co.)
Jamaica	Jamaica Public Service Company Limited (JPS)
Bahamas	Grand Bahama Power Company (GBP) Bahamas Power and Light Ltd. (BPL)

Table 6-2. Utility providers for 16 subject countries

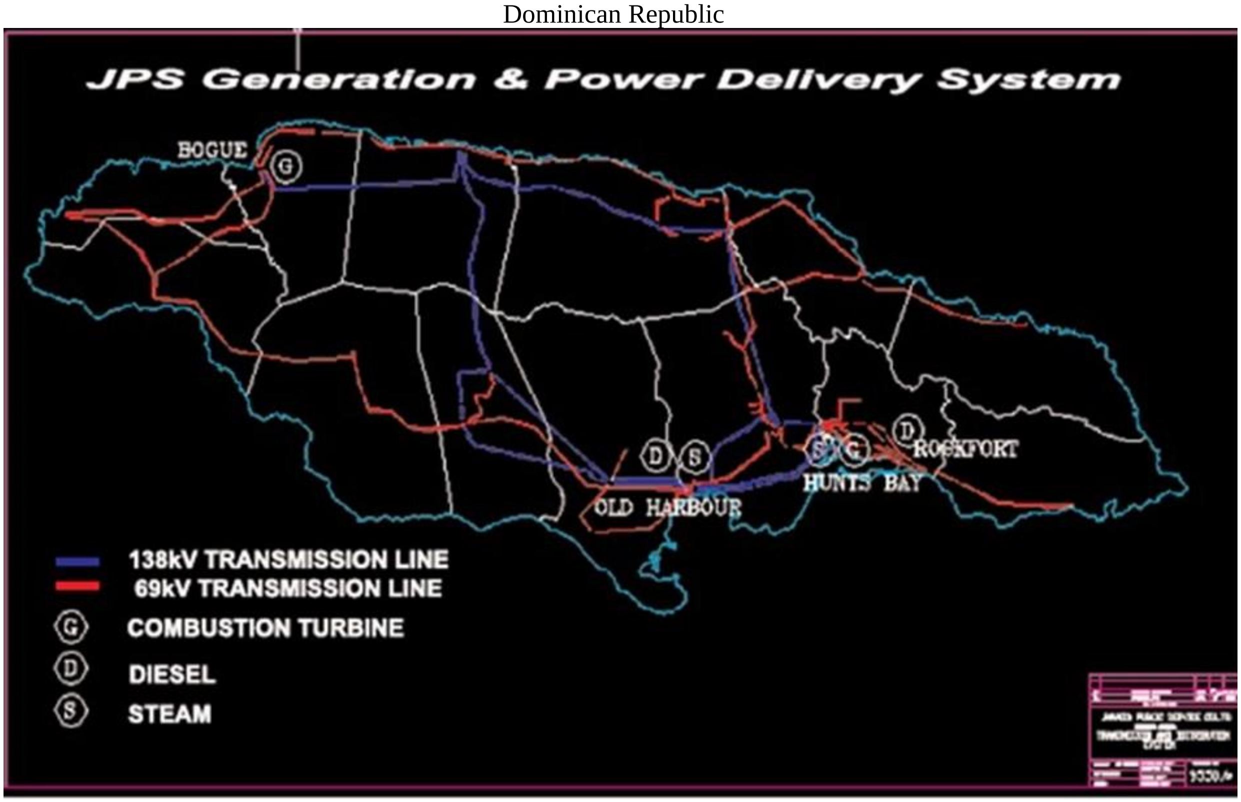
Examples of power distribution maps in the subject countries are presented in Figure 6-3. As discussed previously, the report will focus on substructures and transmission system components of power infrastructure in the Caribbean.



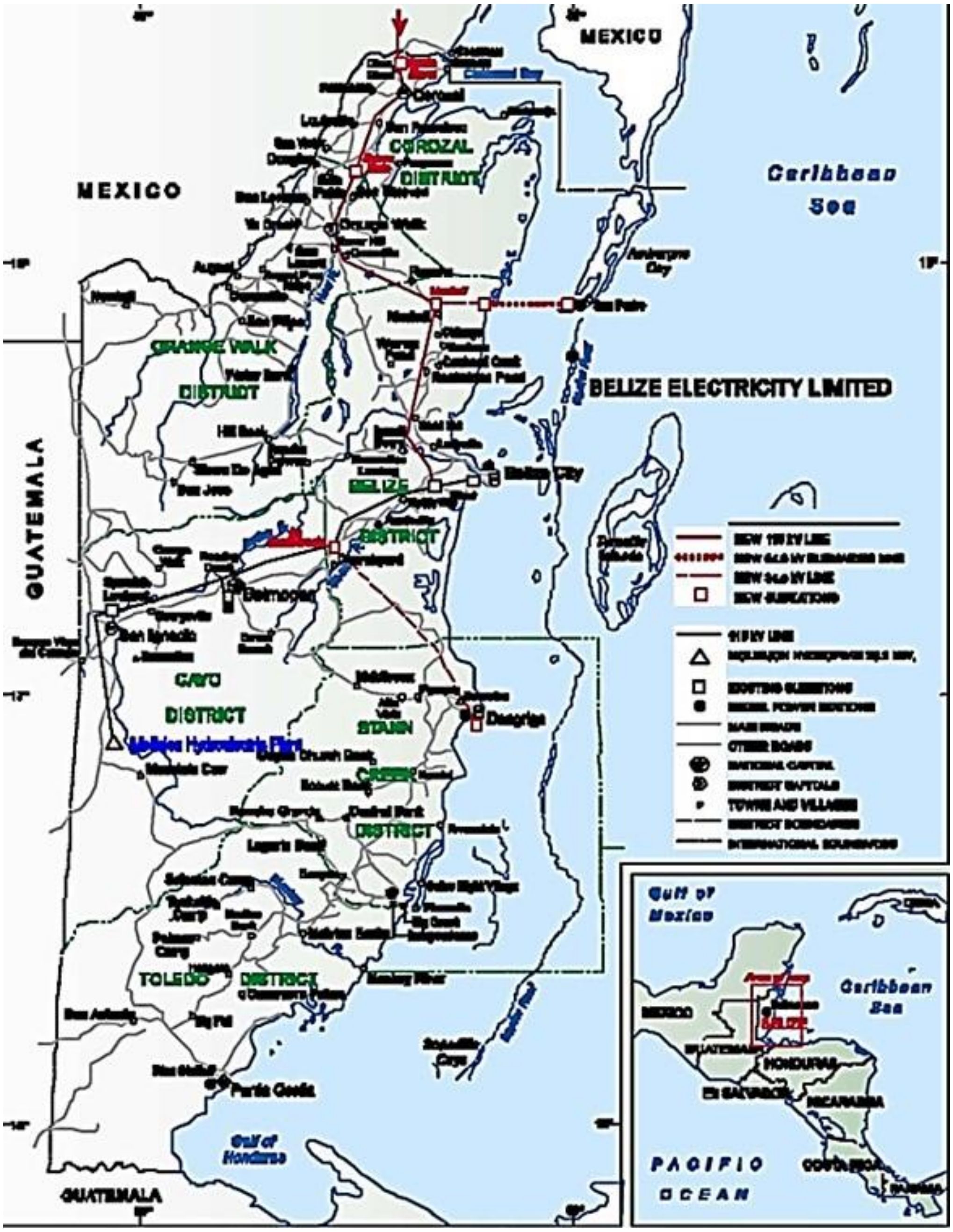
Barbados



Dominican Republic



Jamaica



Belize

Figure 6-3. Power distribution networks (National Energy Grid Map Index, 2020)

Most of the power plants in this region are powered by imported fossil fuels. However, efforts are underway to utilize hydropower, geothermal, solar, and wind power as well; see Figure 6-4.

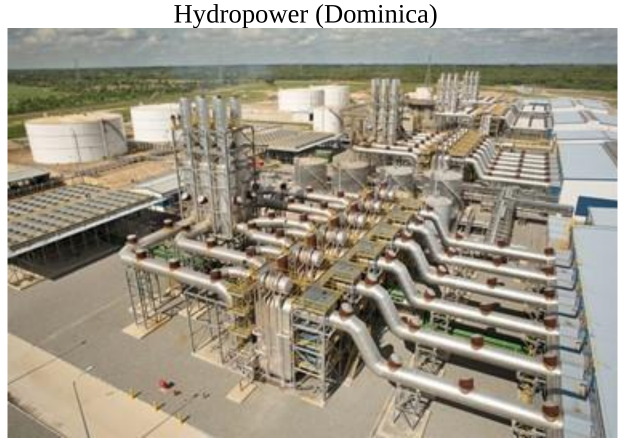
Thermal (oil or diesel) power plant (Suriname)

Geothermal power plant (Dominica)

Wind power (Jamaica)

Hydropower (Dominica)

Solar power (Bahamas)



Natural gas power plant (Dominican Republic)

Figure 6-4. Power generating plants (various sources)

Table 6-3 presents data regarding the power generation facilities, substations, and power distribution for selected countries. In recent years, upgrades to power sector components in several countries have been undertaken or are planned. Examples include:

- Haiti (BU, 2018):

  In recent years, the World Bank has committed to investing $40 million in improving the transmission and distribution network in the PAP area, and the IDB and USAID is spending another $40 million on the rehabilitation of five substations in PAP: Canape Vert, Carrefour Feuille, Toussaint Brave, Croix-des-Bouquets, and Nouveau Delmas.

- Guyana (GPL, 2020):

  [T]hat funds amounting to $2.2 billion were also identified for the rehabilitation of some 55 kilometres of 69 kV transmission line. The distribution system was falling apart in some areas and the project is really about rehabilitation and reconfiguration but there are still generation and transmission aspects and at some point we will be regularly breaking out these projects

- Suriname (EBS, 2020):

  Under the Project, EBS will undertake the upgrade of 36.6 kilometres of sub-transmission and distribution lines, the construction of five new substations, and the expansion or upgrade of three existing substations.

Country	Capacity GWh	Power plants	Substation	Transmission lines
Suriname	1,600	6 (thermal and hydroelectric)	N/A	N/A
Trinidad and Tobago	10,300	N/A	N/A	N/A
Guyana	730	N/A	N/A	N/A
Belize	450	N/A	N/A	3,000 km
Haiti	650	13 (Port-Au-Prince only)	9 (Port-Au-Prince only)	1,700
Dominican Republic	14,400	11 (thermal)	N/A	940
Antigua and Barbuda	330	N/A	N/A	N/A
Dominica	110	3 (hydroelectric)	N/A	N/A
Grenada	200	N/A	N/A	N/A
Saint Kitts and Nevis	210	N/A	N/A	N/A
Saint Lucia	370	1	7	117 km
Saint Vincent and the Grenadines	160	5	N/A	N/A
Sint Maarten		N/A	N/A	N/A
Barbados	1,000	3	13	150 km
Jamaica	4,800	N/A	N/A	N/A
Bahamas	2,200	N/A	N/A	N/A

Table 6-3. Power sector components (various sources)

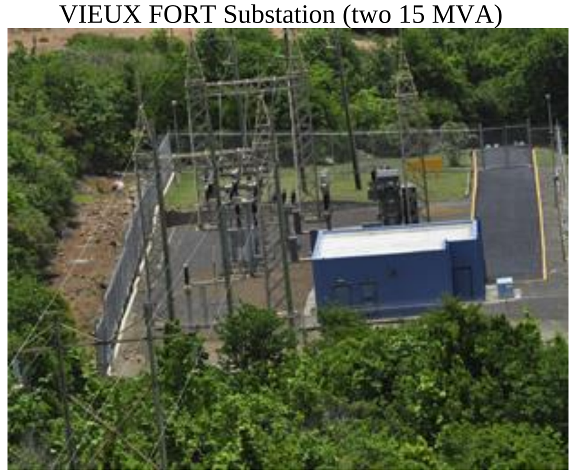
6.3 Typology

Examples of substations in the subject countries are presented in Figure 6-5. As seen in the figures, the substations use similar construction and electrical equipment. Figure 6-6 presents examples of transmission towers. The countries in the Caribbean, similar to practices used worldwide, utilize steel lattice towers for power transmission.

CUL DE SAC Substation (142.5 MVA[13])

VIEUX FORT Substation (two 15 MVA)

REDUIT Substation (two 15 MVA)



PRASLIN Substation (6.5 MVA)

UNION Substation (two 15 MVA)

CASTRIES SUB-STATION (two 15 MVA)

Saint Lucia (adapted from LUCELEC, 2020)

[13] Megavolt-ampere

Duhaney Park Substation New Michelton Halt Substation

Jamaica (adapted from JPS, 2020)

Sophia Substation Sophia Substation

Suriname (adapted from GPL, 2020)

Figure 6-5. Substations in subject countries

Haiti Trinidad and Tobago

Guyana Jamaica

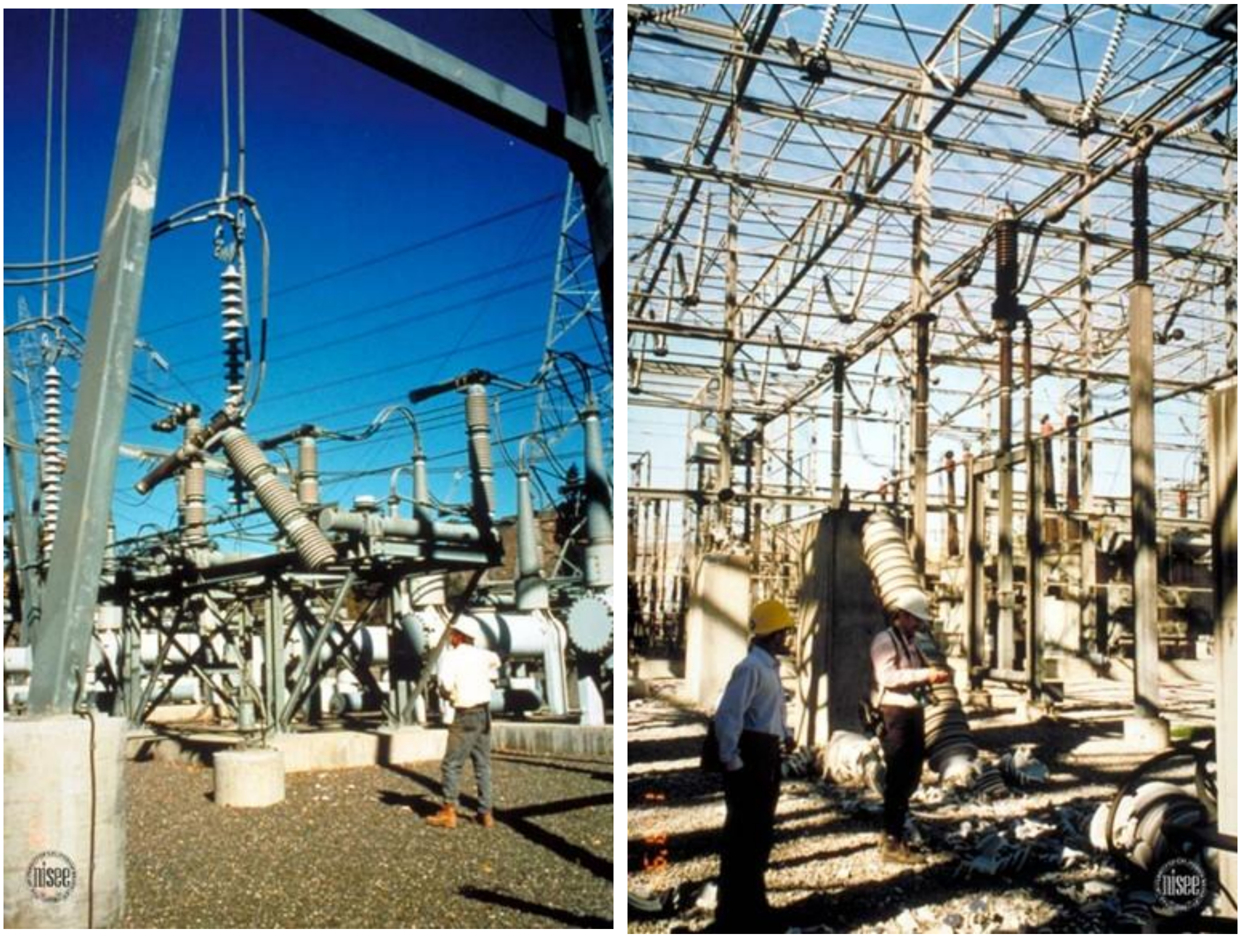
Figure 6-6. Transmission towers and lines in subject countries

6.4 Vulnerable components

Past natural hazards have shown that substations and transmission towers are vulnerable to various natural hazards. Certain components have been known to experience frequent failures. Strengthening of these components would lead to marked improvement in the performance and as such, these components are emphasized in this chapter. Examples of earthquake damage to vulnerable components of substations are presented in Figure 6-7.

Transmission towers are most vulnerable to wind forces, although damage to towers because of earthquake and flooding has also been observed; see Figure 6-8. The strengthening methods presented in this chapter focused on wind loading. However, a retrofitted tower will also perform well in earthquakes.

Falle transformer Rolled transformer



Column shear failure Buckling of slender members

Figure 6-7. Substation damage in past Earthquakes (NISEE, 2020)

Earthquake Ground failure

Hurricane winds	Flood

Figure 6-8. Transmission tower damage as a result of natural hazards

Table 6-4 presents the components that have experienced the most damage in past events and will be considered in this chapter.

Subsector	Type	Component	Vulnerability	Hazard
Power plants	Low-rise RC	Design	Nonductile construction	Earthquake
		Grade construction	No flood protection	Flood
		Nonstructural components	Lack of anchorage	Earthquake
		Foundation	Shallow	Earthquake, Liquefaction
Substations	Control building	Design construction	Strength stiffness	Earthquake
		Grade construction	No flood protection	Flood
		Battery racks, cable trays, etc.	Lack of anchorage/bracing	Earthquake
		Foundation	Shallow	Earthquake, Liquefaction
	Transformers	Anchorage	No or inadequate anchorage	Earthquake
	Frames	Steel members	Slender low damped	Earthquake, wind
	Equipment	High-voltage units	Not qualified	Earthquake
Transmission towers	Lattice towers	Tower	Lack of strength stiffness	Earthquake, wind
		Foundation	Inadequate capacity	Flood, Liquefaction
		Lines	Vibration. Low damping	Wind

Table 6-4. Vulnerable components of power infrastructure and corresponding hazards

6.5 Design practice

6.5.1 Overview

ASCE 113 (2007) provides design criteria for structures. Although intended for the U.S. and North America, the document is used worldwide, including in the Caribbean. ASCE 113 wind design is based on provisions of ASCE 7. Similarly, the Pan American Health Organization (PAHO) has developed wind speeds to be used with wind provisions of ASCE 7 (2019). The key provisions of the standard are summarized below.

6.5.2 Wind design

The wind force is calculated based on the wind speed, terrain, importance factor, gust factor, and projected surface area. Typically, either a 50- or 100-year return interval is used to compute wind loading. The effect of wind flow in any direction is considered in design. In hurricane-prone zones, deflection limits are set for utilities for a more frequent event (such as a 5-year wind velocity). When wind vibrations are of concern, the components should be evaluated and designed for forces from such vibrations.

6.5.3 Earthquake loading

Site-specific earthquake loading is defined based on site seismicity and soil condition. Typically, a 500-year event is used for design. Both horizontal components and vertical components (equal to 80%) are considered in design.

6.5.4 Flood loading

Infrastructure located in coastal zones needs to be designed for several types of floods. Structures or equipment subject to differential pressure from water should be designed to account for this force.

6.5.5 Load combinations

The load combinations for design that include natural hazard forces are listed in Table 6-5. Note that for coastal areas, like many of the zones for the subject buildings, a combination of wind and flood loading is applied to structures and components.

Case	Dead	Operating	Short circuit	Wire tension	Wind	Earthquake	Flood
I	1.1	1.0	0.75	1.1	1.2	0	2.0
III	1.1	1.0	1.0	1.1	0	0	1.0
IV	1.1	1.0	0.75	1.1	0	1.25	0

Table 6-5. Design load combinations (adapted from ASCE, 2007)

6.6 Fragility functions

6.6.1 Earthquake fragility functions based on FEMA Hazus methodology

6.6.1.1 Overview

In this section, physical (structural) damage and operation loss (functionality) fragility relations for substations and transmission tower types common in the Caribbean are presented. For the structures, two types of design are considered: **conventional** (non-seismic design constructed prior to adoption of seismic codes) and **seismic** (constructed after adoption of seismic provisions in design).

6.6.1.2 Damage states

FEMA Hazus (FEMA, 2003a) defines a number of damage states for substations. The damage states relevant to the Caribbean are summarized in Table 6-6.

Damage state	State	Description
DS1	None	No observable damage
DS2	Slight	Failure of 5% of the disconnect switches (i.e., misalignment), or the failure of 5 % of the circuit breakers (i.e., circuit breaker phase sliding off its pad, circuit breaker tipping over, or interrupter head falling to the ground), or by the building being in minor damage state
DS3	Moderate	the failure of 40% of disconnect switches (e.g., misalignment), or 40% of circuit breakers (e.g., circuit breaker phase sliding off its pad, circuit breaker tipping over, or interrupter-head falling to the ground), or failure of 40% of current transformers (e.g., oil leaking from transformers, porcelain cracked), or by the building being in moderate damage state.

Damage state	State	Description
DS4	Extensive	Failure of 70% of disconnect switches (e.g., misalignment), 70% of circuit breakers, 70% of current transformers (e.g., oil leaking from transformers, porcelain cracked), or by failure of 70% of transformers (e.g., leakage of transformer radiators), or by the building being in extensive damage state.
DS5	Complete	Failure of all disconnect switches, all circuit breakers, all transformers, or all current transformers, or by the building being in complete damage state.

Table 6-6. Substation damage states (adapted from FEMA, 2003a)

6.6.1.3 Fragility functions for ground shaking

Based on whether a substation is constructed to meet seismic requirements, FEMA Hazus (FEMA, 2003a) has developed fragility functions for substations corresponding to the probability of exceeding a damage state. Fragility functions relevant to common bridges in the Caribbean are summarized in Table 6-7.

	Mean, PGA, g				Standard deviation, g
Typology	DS2	DS3	DS4	DS5	
Stand components	0.1	0.2	0.3	0.5	0.4-0.6
Seismic components	0.15	0.25	0.35	0.7	0.4-0.6

Table 6-7. Substations fragility parameters, ground motion (adapted from FEMA, 2003a)

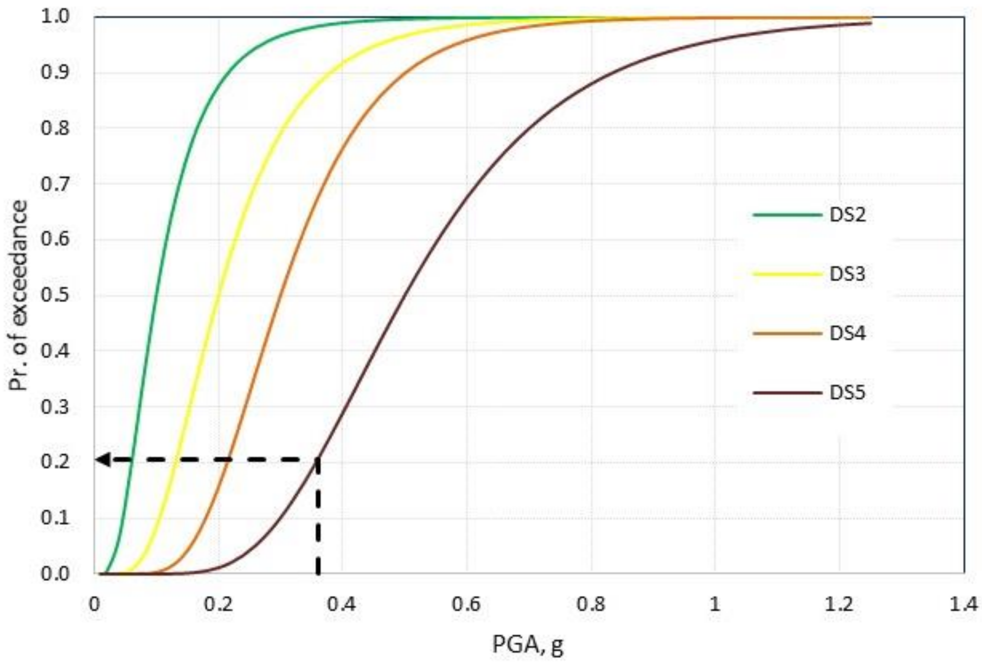
6.6.1.4 Fragility functions: ground failure (liquefaction)

The fragility functions for ground failure hazard are similar to what was presented for bridge or building foundations.

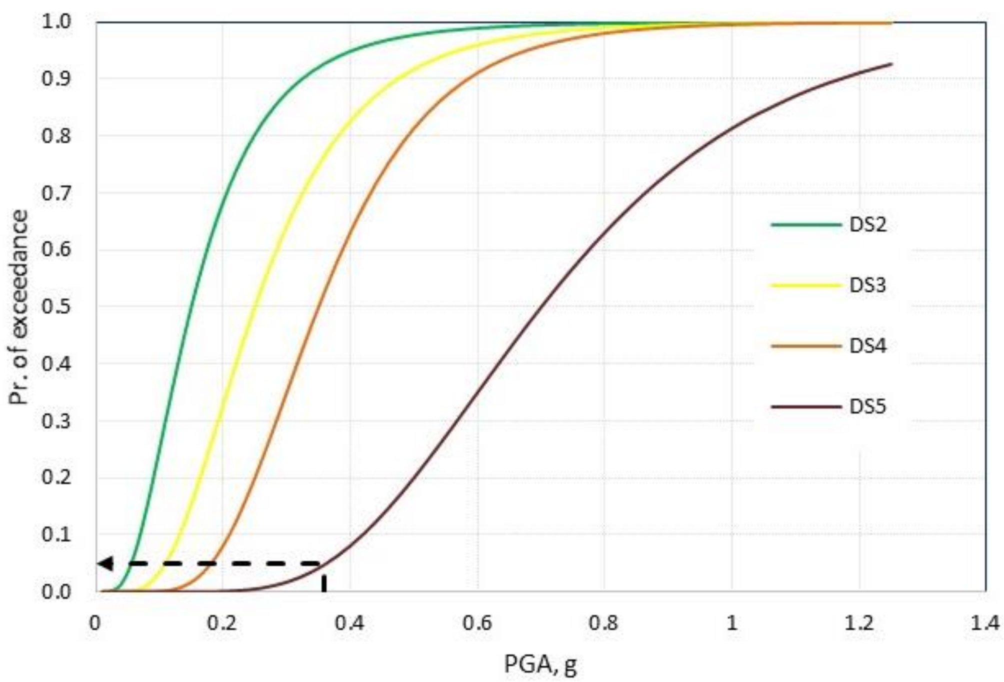
6.6.1.5 Typical substation

Figure 6-9 presents the fragility functions for two substations, one with conventional design and one with properly-designed anchorage and seismic components. Also shown in the figure is the line with ordinates of approximately 0.36 g, which corresponds to a moderate earthquake. As seen in the figures, the probability of experiencing significant damage is reduced measurably when retrofit measures are implemented.

Figure 6-10 presents the probability of the substation being in a given damage state when subjected to a moderate earthquake. Note that once seismic retrofitting is implemented, the probability of the substation experiencing the detrimental DS5 state is reduced from approximately 20% to approximately 5%.



Existing



Retrofitted

Figure 6-9. Fragility functions for example substation

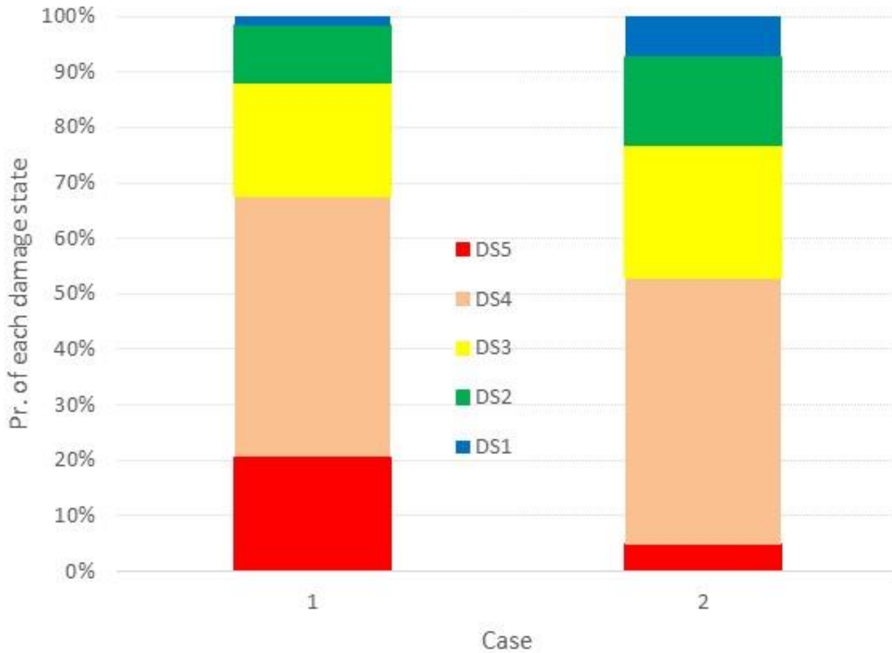
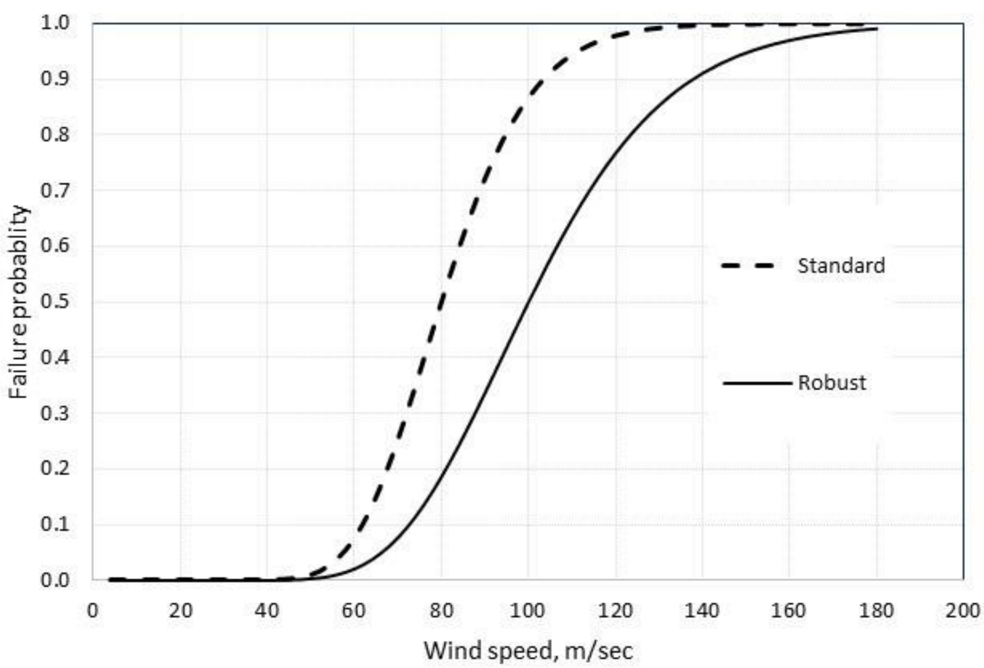


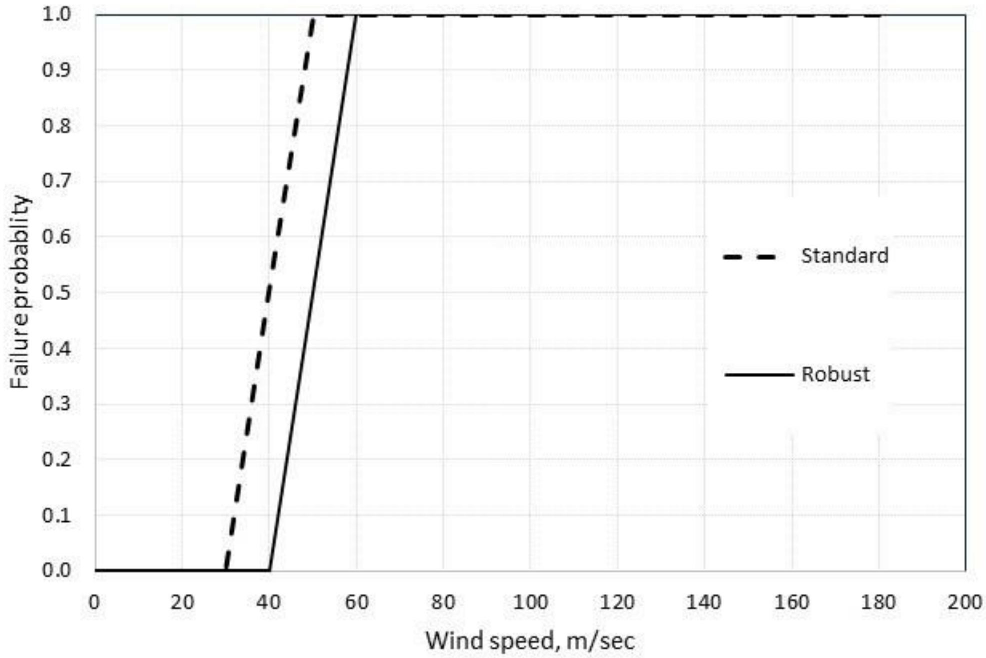
Figure 6-10. Distribution of damage states

6.6.2 Wind fragility functions

Panteli et al. (2017) conducted numerical studies and developed tower fragility functions for typical transmission towers subject to wind loading. They considered two cases: stand design and robust or enhanced design. As seen in Figure 6-11, the tower's probability of failure is reduced from approximately 50% to 20% for a 50 m/sec windstorm. Similarly, when enhanced design is employed, the failure probability of the transmission line is significantly reduced.



Transmission tower



Transmission line

Figure 6-11. Fragility functions for typical transmission towers (adapted from Panteli et al., 2017)

6.7 Strengthening techniques

6.7.1 Substations

Examples of strengthening techniques for substations are presented in this section. The emphasis is on the components that have been the most vulnerable in past earthquakes.

6.7.1.1 Use of seismically qualified components

The Institute of Electrical and Electronics Engineers (IEEE) 693 provides seismic requirements for substation components (2018). IEEE 693 requires that the qualification level for substation components be established based on the site-specific PGA; see Table 6-8.

Qualification level	Site PGA, g
Low	≤0.10
Moderate	≤0.50
High	>0.5

Table 6-8. Substation components qualification levels (adapted from IEEE, 2018)

There are specific prescriptive requirements for each qualification level. Many of the subject countries in the Caribbean will likely fall in the “moderate” qualification level.

6.7.1.2 Redundancy

To improve the resiliency of the power infrastructure, a network with redundant components is preferred. The redundancy can be achieved in two important ways:

- Employ different design criteria for two substations on the same network. For example, one substation can use more rigid components and one, more flexible ones. This ensures that there is less likelihood that both substations will be damaged in a given earthquake and that both can maintain the power supply (at least partially) until the damaged substation is repaired.
- Ensure spare parts are stored at substations. In the event of damage in an earthquake, the damaged component can be replaced with the spare part and service restored rapidly.

Clearly, the utilities sector needs to balance the cost of spare parts with the utility of having such parts. The Electric Power Research Institute (EPRI) presents a transformer spare strategy matrix (2014); see Table 6-9. The data correspond to action taken in preparation before an event like a large earthquake.

Inventory Creation Scenarios	Cost	Manufacturing Time	Availability Timing	Reliability	Protection time
Priority	Low	Low	High	Low	High
Utility orders spares for transformers (nearing end of life) earlier than needed	Moderate	Standard	Fast	High	Short
Utility orders spares for any critical transformers (regardless of remaining life)	High	Standard	Fast	High	Long
Utility keeps retired transformers on hand	Low	None	Fast	Low	Short
Utility participates in sharing arrangements	Low	None	Fast	High	Long
Utility orders emergency spare transformers before the event for its own use only	High	Short	Fast	High	Long
Utilities pool resources to order shared spares (emergency spare transformers)	Moderate	Short	Fast	High	Long

Table 6-9. Transformer spare strategy matrix (adapted from EPRI, 2014)

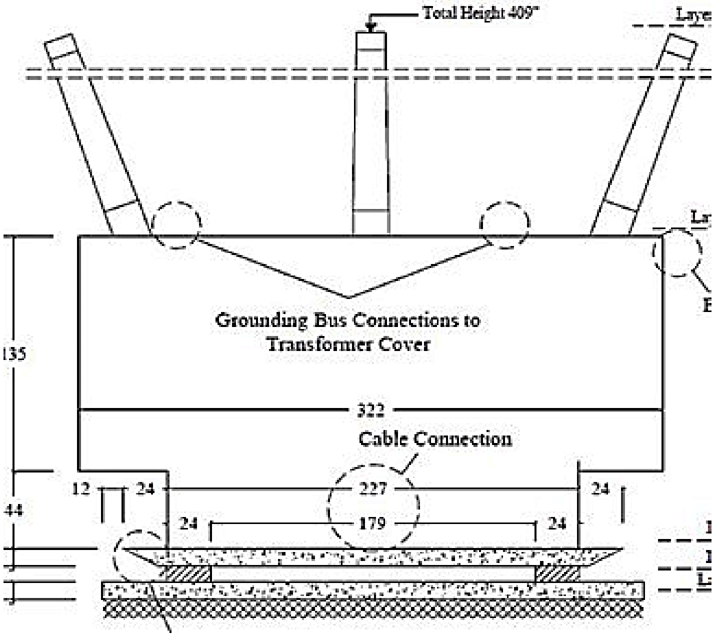
6.7.1.3 Transformers

As discussed previously, one of the common failure modes for transformers has been sliding or overturning due to inadequate anchorage. Depending on the type of mounting, the anchorage for existing transformers can be significantly improved by addition of steel members and connections. Figure 6-12 (Knight and Kempner, 2004) presents samples of improvement measures for two types of common installations.

Rail installation Pad installation

Figure 6-12. Typical transformer anchorage enhancements (Knight and Kempner, 2004)

Transformers – the most important and expensive part of the transmission system and one of the most vulnerable components – are heavy and relatively rigid units and thus are ideal candidates for the use of base isolation. Kempner and Riley (2016) write that the Bonneville Power Administration (BPA) has recently incorporated base isolation for some of their transformers; see Figure 6-13.

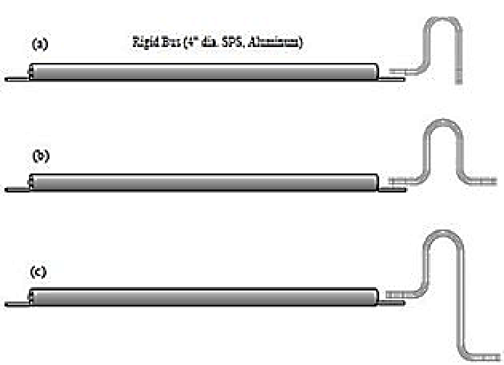


Plan Installation

Figure 6-13. Seismic base isolation of transformers (Kempner and Riley, 2016)

6.7.1.4 Flexible connectors

In substations, electrical conductivity is achieved by use of connectors (bus) between two components. Since the two equipment pieces being connected typically have different vibration frequencies, they do not move in-phase (in unison) and this results in force being developed in the bus. This connector force is resisted by the interconnected equipment and thus induces additional demand on the components, which can result in their failure. This is primarily the case where the bus is rigid, since this configuration has a limited capability of allowing differential motion. To improve seismic performance, flexible ends can be added to rigid bus connections to accommodate this differential movement and thus mitigate the connector forces acting on the equipment. Examples are shown in Figure 6-14 (Song and Der Kiureghian, 2004).

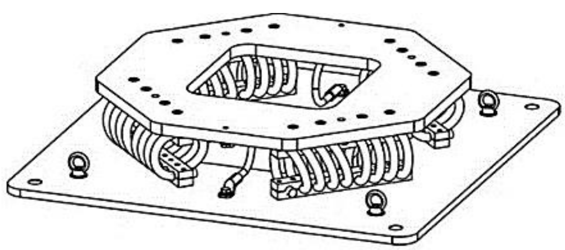


Plan Installation

Figure 6-14. Retrofit of rigid bus by flexible connectors (Song and Der Kiureghian, 2004)

6.7.1.5 Wire rope isolation

Circuit breakers are a key component of substations. These are tall units with heavy mass on top supported on slender steel structures; they have suffered failures in past earthquakes. Testing of rigidly-mounted units has shown the failure that could be expected in earthquakes. Wire ropes can be used as a means to enhance the seismic performance of circuit breakers. The wire ropes allow rocking motion at the base of the component and thus reduce the seismic forces. Examples are shown in Figure 6-15 (Alessandri et al., 2015).



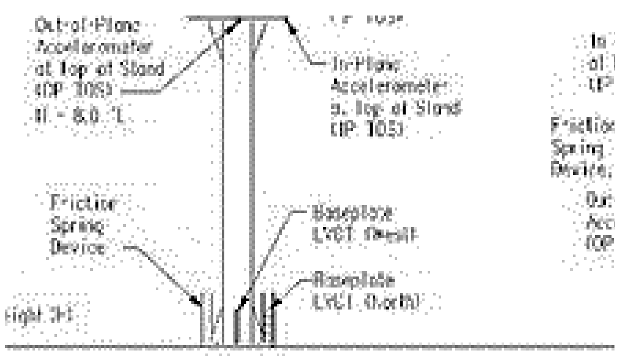
Plan

Installation

Figure 6-15. Seismic protection of circuit breakers, wire ropes (Alessandri et al., 2015)

6.7.1.6 Spring dampers

As discussed previously, most equipment in substations is supported on stands. An approach to reducing the seismic demand on the stand and the equipment is the use of friction spring devices with self-centering capabilities. The passive dampers allow rocking motion at the base of the component and dissipate seismic energy through friction. Examples are shown in Figure 6-16.



Plan

Installation

Figure 6-16. Seismic protection by means of spring dampers (Riley et al., 2006)

6.7.1.7 Retrofit design of elevated supports

Many components, such as discontent switches, are mounted on tall, elevated frames. Often, these frames are flexible and thus amplify the motion imparted on the equipment. Adding horizontal and vertical bracing to these frames will increase their stiffness significantly and thus reduce the motion and forces experienced by the supported equipment. An example is presented in Figure 6-17 (Gilani et al., 2000). The retrofitted frame does not amplify the response. However, for the existing frame, the amplification factor is over 2.

Exiting frame

Retrofitted frame

Figure 6-17. Seismic protection by means of improved support design (Gilani et al., 2000)

6.7.1.8 Composite insulators

Porcelain has been used extensively for insulation in substations. It is relatively inexpensive, durable, and easily available. However, porcelain is also quite brittle and has experienced significant failure in past earthquakes. Composites can be used in place of porcelain to act as insulators. Composite insulators are significantly lighter (thus reducing seismic forces) and are not susceptible to brittle failure modes, such as fractures. Composite insulators can be used for a variety of components, such as bushings, disconnect switches, or any other element with insulators. An example is presented in Figure 6-18 (Gilani et al., 2000).

Porcelain insulators

Composite insulators

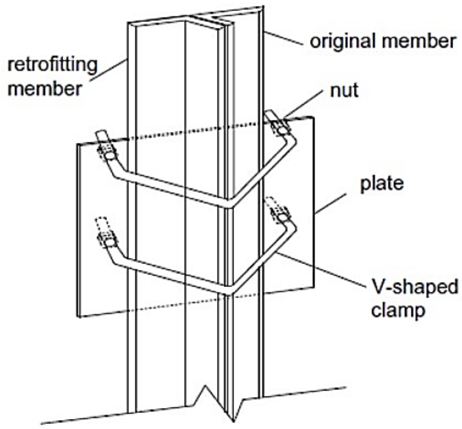
Figure 6-18. Seismic protection by use of composite insulators design (Gilani et al., 2000)

6.7.2 Transmission towers and lines

Transmission towers use slender steel lattice members. In particular, the diagonal members are typically single- or double-angle members with low buckling capacity. Transmission lines are subject to failure due to excessive wind-induced vibrations. The tower system has low structural damping, and this contributes to the vulnerability.

6.7.2.1 Retrofit of transmission towers

Xie and Zhange (2020) developed a method for retrofit of the diagonal members of transmission towers while maintaining service; see Figure 6-19. They noted that by clamping plates at regular intervals to the diagonal members, the capacity was increased by close to 80%.



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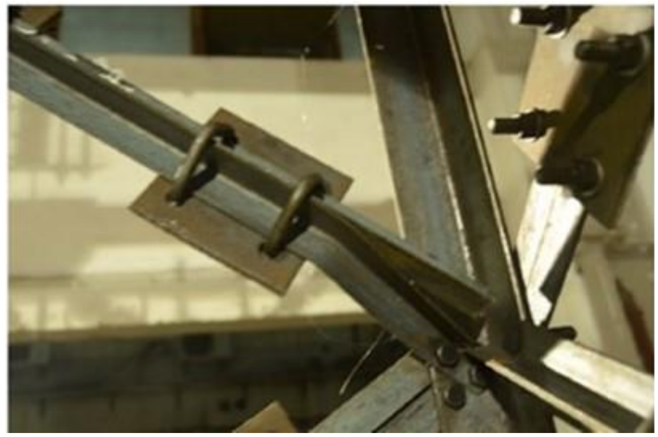
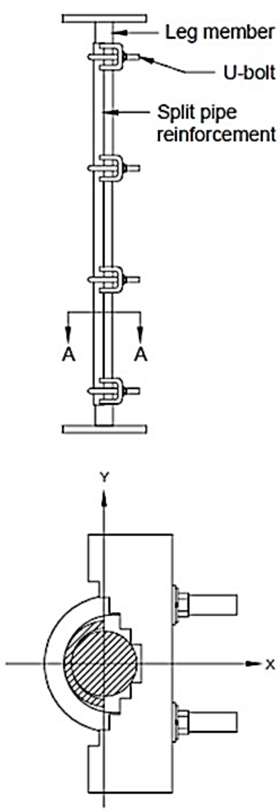


Installation

Figure 6-19. Strengthening of existing transmission tower diagonal members (Xie and Zhange, 2020)

Kumalasari et al. (2005) studied the retrofitting of latticed towers with split pipe sections attached to the tower legs with U-bolts; see Figure 6-20. On average, the load carrying capacity was improved by approximately 50%.



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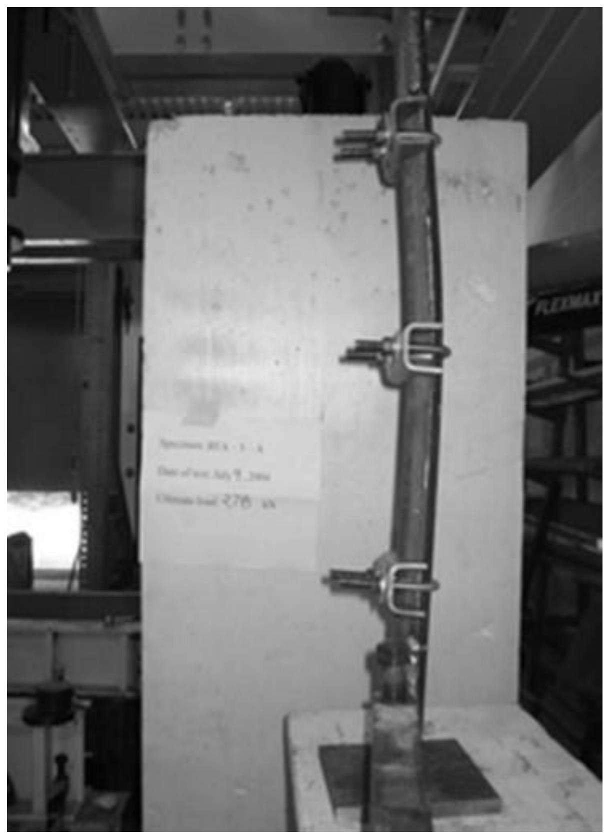

Installation

Figure 6-20. Strengthening of existing transmission tower leg members (Kumalasari et al., 2005)

Trovato et al. (2015) examined the use of retrofitting slender members by restraining tubes to provide resistance to buckling during earthquakes or hurricanes; see Figure 6-21. It was noted that the proposed solution prevented buckling of the tower members, increased the capacity by nearly 70%, and maintained the load carrying capacity after reaching its capacity.

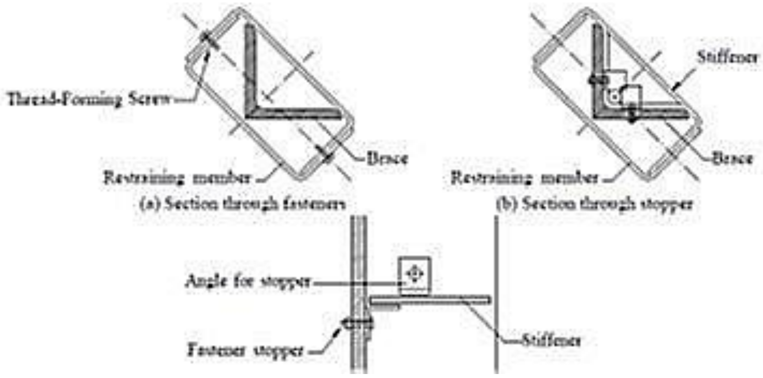

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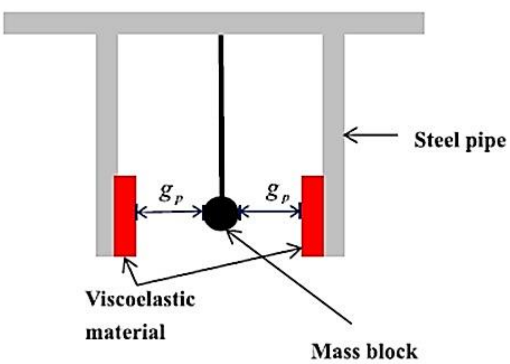
Figure 6-21. Buckling prevention of existing transmission tower leg members (Trovato et al., 2015)

Zhang et al. (2013) studied the use of tuned mass dampers (TMDs) to increase damping in towers and reduce the motion and stresses in the tower members; see Figure 6-22. For low-damping systems, they obtained a vibration reduction of 26%.



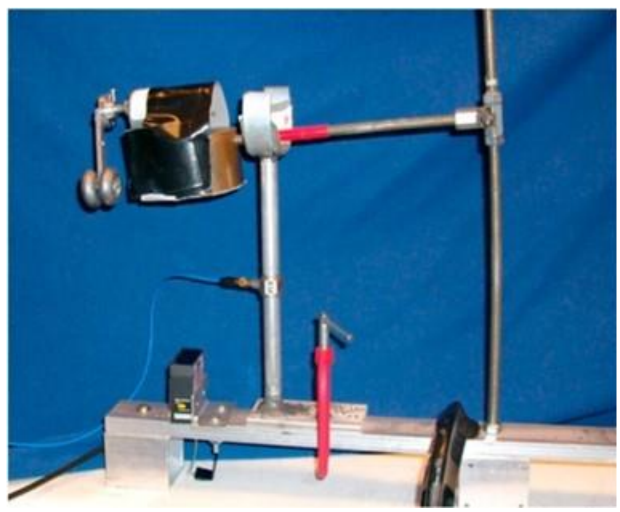


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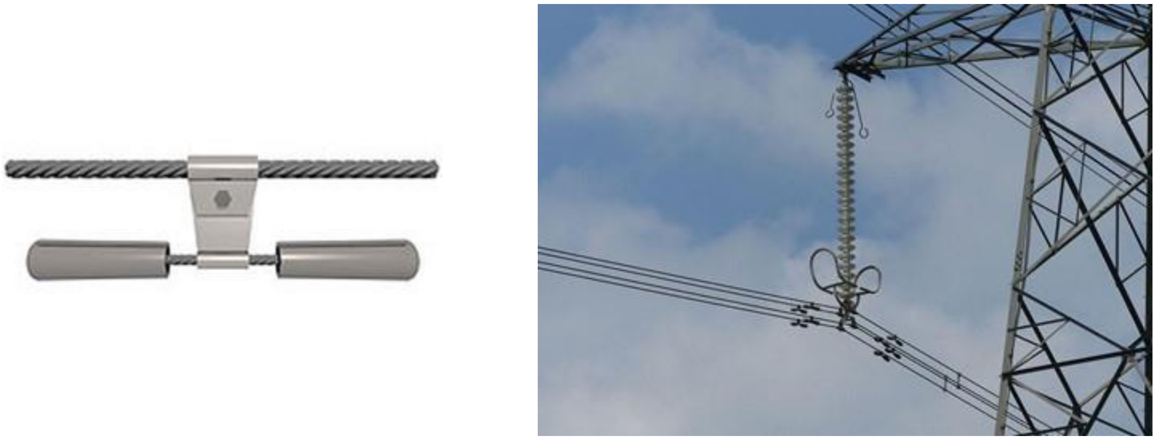
Figure 6-22. Tuned mass damper for transmission tower (Zhang et al., 2013)

6.7.2.2 Retrofit of transmission lines

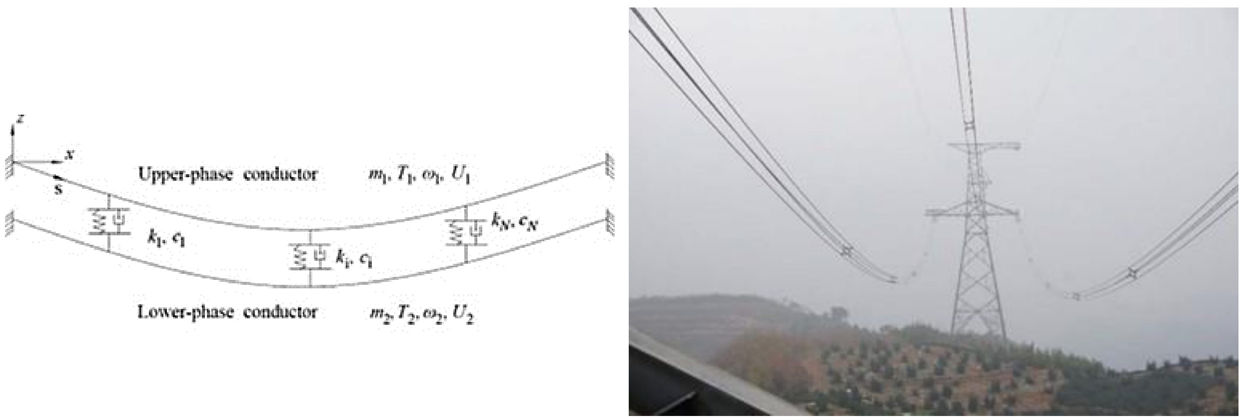
Stockbridge dampers are small TMDs attached to individual transmission lines that are used to suppress the vibration of transmission lines; see Figure 6-23. The installation of Stockbridge dampers reduces the vibrations in cables and by changing the dynamic properties of the cable, serves to suppress motions in the predominant natural frequencies. Since the most vulnerable segment of an individual cable is at the clamping ends, typically two dampers are installed per span. Dampers are typically used to suppression motion in vertical plane where motion is the most severe.



Plan Installation

Figure 6-23. Stockbridge dampers for transmission line (various sources)

Lou et al. (2020) have studied an innovative method of using viscoelastic dampers to interconnect the individual cables as a means of wind mitigation; see Figure 6-24.

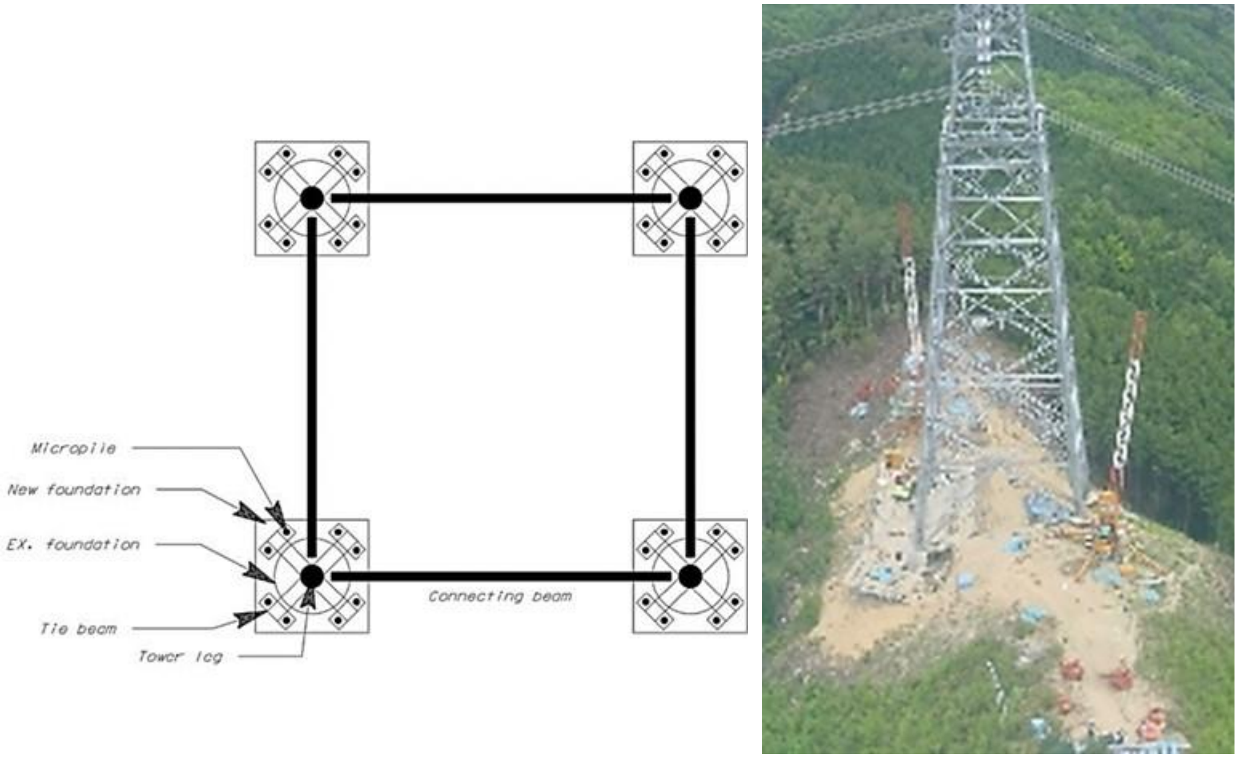


Plan Installation

Figure 6-24. Viscoelastic dampers for transmission line (Lou et al., 2020)

6.7.3 Foundation retrofit

The retrofit of foundations for substation buildings follows the same procedure as discussed for buildings. For substation equipment, typically no foundation upgrades are required; instead, equipment anchorage as discussed previously might require strengthening. When flood or liquefaction is of concern, it is simpler to relocate the equipment or provide flood protection. The transmission tower, on the other hand, could be a vulnerable component that might require strengthening. For towers located in soft soil or in flood plains, foundation retrofit incorporating micropiles can be implemented that does not require service interruption. An example is shown in Figure 6-25 (Yamane, 2020).



Plan Installation

Figure 6-25. Retrofit of transmission tower foundations (Yamane, 2020)

6.8 Cost analysis

6.8.1 Introduction

Electrical utilities face competing economic constraints. To implement strengthening techniques, it is critical to undertake a comprehensive plan that includes the following components:

- Perform a screening of all infrastructure and detail their vulnerabilities.
- Prioritize the most vulnerable components and schedule them for upgrade
- Develop a cost estimate for strengthening
- Perform a feasibility study on the benefits of the strengthening

It is noted that most cost estimates are for the United States. However, it is anticipated that the ratio of strengthening to reconstruction costs are similar in the U.S. and the Caribbean.

6.8.2 Reconstruction costs

6.8.2.1 Capital cost

Black & Veatch (2012) conducted a study regarding the capital costs for substations and transmissions. They examined different voltage classes, pole structures and transmission line length, terrain, and location. Findings from their work for the following system are presented in this section.

- 230 kV single circuit
- 4 lines
- 30 megavolt ampere (MVA)
- 10 km of transmission line
- No cost for land acquisition because reconstruction of substations and towers are at existing locations

- No allowance for terrain is made because either reconstruction or retrofit would have similar constraints.

Table 6-10 and Table 6-11 show typical substation and transmission cost calculations, respectively, for the selected example. Note the significant cost associated with reconstruction.

Baseline cost	$1,650,000
Cost per line	$1,440,000
Breaker multiplier	1.5
Lines	4
Transformer	$800,000
Shunt reactor	$600,000
Series Capacitor	$900,000
Transformer cost	$13,000,000

Table 6-10. New substation cost data for 230 kV system (adapted from Black & Veatch, 2012)

Baseline cost	Per km	$5800,000
Conductor	Type	Aluminum Conductor Steel Supported (ACSS)
	Multiplier	1.08
Transmission tower	Type	Lattice
	Multiplier	0.9
Transmission line	Length	5 to 20 km
	Multiplier	1.2
Transmission cost		$6,800,000

Table 6-11. New transmission cost data for 230 kV system (adapted from Black & Veatch, 2012)

6.8.2.2 Component cost

Midcontinent Independent System Operator (MISO) has developed cost estimates for new construction of expansion of the energy infrastructure (2020). The cost estimates include both overhead and allowance for funds used during construction. Since these could vary between the U.S. and Caribbean, these costs are not included in this section. Findings from their work for the following system are presented in this section.

- 230 kV single circuit
- 4 lines
- 30 MVA
- 10 km of transmission line
- Towers are 300 m apart
- No cost for land acquisition because reconstruction of substations and towers are at existing locations
- No allowance for terrain is made because either reconstruction or retrofit would have similar constraints.
- The cost data for typical substations, transmission towers, and transmission lines are presented in Table 6-12, Table 6-13, and Table 6-14, respectively. The cost for removal or rebuilding of transmission lines is presented in Table 6-15. Note the significant cost associated with reconstruction of power infrastructure that could be necessary after a major natural hazard event.

Component	Cost item	Cost
Circuit breaker	Material cost	$97,000
	Installation cost	$10,000
	Wiring cost	$15,000
	Foundation cost (6 m^3)	$10,000
	Cost per unit	$134,000
Disconnect switch	Material cost	$21,000

	Installation cost	$10,000
	Wiring cost	$8,000
	Steel stand (1700 kg)	$17,000
	Foundation cost (6 m^3)	$10,000
	Cost per unit	$66,000
Voltage transformer	Material cost	$36,000
	Installation cost	$3,000
	Wiring cost	$12,000
	Steel stand (800 kg)	$9,000
	Foundation cost (3 m^3)	$5,000
	Cost per unit	$64,000
Current transformer	Material cost	$129,000
	Installation cost	$3,000
	Wiring cost	$12,000
	Steel stand (800 kg)	$9,000
	Foundation cost (3 m^3)	$5,000
	Cost per unit	$158,000
Bus and fittings	Material cost	$9,000
	Installation cost	$11,000
	Wiring cost	--
	Steel stand (1,000 kg)	$10,000
	Foundation cost (5 m^3)	$10,000
	Cost per unit	$40,000
Transformer	$8000 /MVA	$240,000

Table 6-12. New substation cost data for 230 kV system (adapted from MISO, 2020)

Steel, kg	7,000
Concrete foundation, m^3	18
Material cost	$29,000
Installation cost	$43,000
Hardware cost	$7,000
Foundation cost	$33,000
Total cost/tower	$111,000
Total cost	$3,700,000

Table 6-13. New transmission tower cost data for 230 kV system (adapted from MISO, 2020)

Type	ACSS
Size	1,000 mm^2
Installation cost/ m	$18
Material cost/ m	$14
Accessories cost/ m	$1
Total cost/m	$33
Total cost	$330,000

Table 6-14. New transmission line cost data for 230 kV system (adapted from MISO, 2020)

Transmission line removal/km	$170,000
Transmission line rebuild/km	$900,000

Table 6-15. Transmission line removal and rebuild cost data (adapted from MISO, 2020)

6.8.2.3 Substation cost estimator

There are a number of tools available to estimate the capital cost for substations. The reconstruction cost for substations can be computed by computing the estimated number of each type of component that would need to be replaced after a natural disaster by the unit costs for the component. An example is presented in Table 6-16 (PEguru, 2020).

Component	Description	Cost
Transformer	10MVA to 50MVA medium power transformer	$ 800,000
Circuit breaker	230kV 3000 AMP breaker	$ 165,000
Circuit switcher	230 kV 3000 Amp switcher	$ 85,000
Disconnect switch	230 kV 3000 Amp switch motor operator	$ 33,000
Capacitor	230 kV 100MVAR neutral reactor and stand	$ 250,000
Voltage transformer	230Kv RELAY GRADE	$ 9,000
Current transformer	138kV transformer	$ 15,000
Surge arresters	230 kV/180 MCOV porcelain arrester	$ 10,500
Service transformer	138kV transformer	$ 150,000
Carrier equipment	230Kv TUNER	$ 25,000
Insulators	230 kV rated insulator	$ 6,000
Control building	20x8 m for 20 panels	$ 1,500,000
Transmission structure	230 kV A-frame dead-end 30 m tall	$ 560,000
Control cables	#6	$20/m
Power cables	500 mm^2	$1500/m
Bus	100 mm Aluminum tube	$100/m

Table 6-16. Examples of component unit cost for substation components (PEguru, 2020)

6.8.3 Strengthening costs

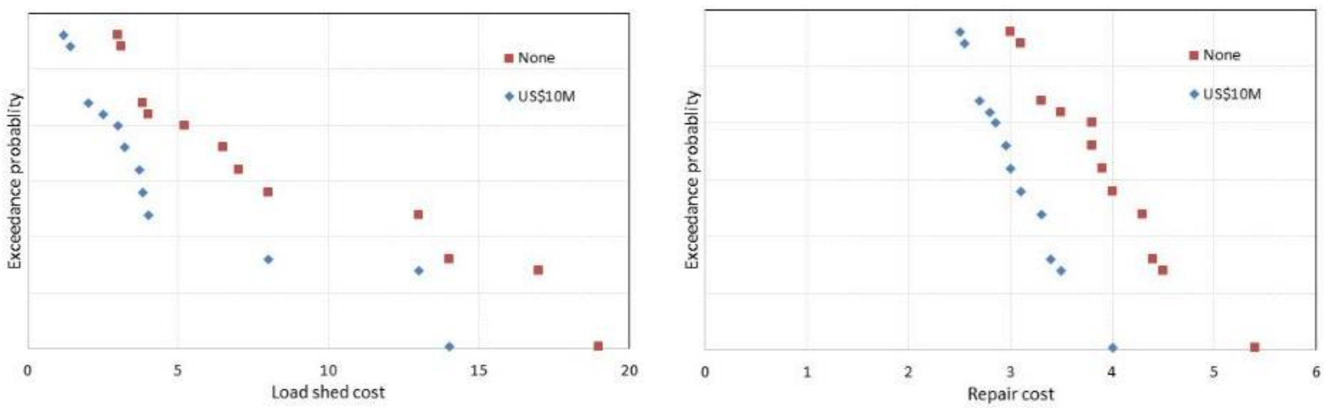
6.8.3.1 Substations

Romero et al. (2013) examined the cost effectiveness of seismic retrofitting of substations in the New Madrid seismic zone of the Central U.S. They examined four cases; see Table 6-17. Examining the cases of “no mitigation” and a mitigation with capital investment of US$10 million, they noted (see Figure 6-26):

- The US$10 million mitigation investment reduces the load shed costs and repair costs to approximately 40% and 20%, respectively, of no mitigation cost.
- The savings for this case translates to approximately US$12 million annually from an initial investment of US$10 million.

Retrofit cost	Substation anchorage
None	0%
US$5 million	60%
US$10 million	87%
US$20 million	98%
US$50 million	100%

Table 6-17. Substation retrofit scenarios (adapted from Romero et al., 2013)



Load shed cost Repair cost

Figure 6-26. Retrofit effectiveness of substations (adapted from Romero et al., 2013)

6.8.3.2 Transmission towers

BPA (2011) has developed a new design for single- and double-circuit transmission towers. The towers use cross-shaped members that are much stronger than typical single angle or double angle members; see Figure 6-27. The new towers, although stronger than traditional design, use less steel and thus can result in savings of up to a quarter of the tower's cost. Additionally, the towers can be placed more than 300 m apart, which is the typical spacing of traditional towers. The towers are designed for extreme events and can withstand wind speeds of 160 km/hr or more.

Tower member Constructed tower

Figure 6-27. New transmission tower design (BPA, 2011)

Considering the hypothetical example case of 10 km of transmission line, for U.S. application, the typical traditional towers cost of approximately US$100,000 is used for smaller single circuit lines. The cost of the new tower is estimated at US$75,000 per unit. As noted in Table 6-18, the retrofitting cost pays for itself after the first major hurricane impacting the area. Given the propensity of Caribbean countries to experience hurricanes and windstorms on an annual basis, during the design life, this or other retrofitting methods can result in significant cost savings. Note the cost savings do not include the additional savings that result from fees collected because the system has remained operational.

Parameters	As-is	Upgraded
Configuration	Double circuit	Double circuit
Length, km	10	10
No of towers	34	20
Initial upgrade cost	0	US$1.5million
Event	Cat III or above hurricane	
Towers damaged	17 (50%)	0
Replacement cost	US$1.7million	0

Table 6-18. Example of transmission tower upgrade cost

6.8.3.3 Transmission lines

Stockbridge dampers cost between US$2-5 for the quantities typically ordered by the utilities. Including installation, a conservative cost of US$20 per damper is assumed here. The cost of line replacement is assumed as US$30/m. As noted in Table 6-19, the improvements pay for themself after the first major hurricane impacting the area. This savings is multiplied through the years, as the system would be resilient and can withstand repeated wind events. When the loss of revenue from customers due to downtime is added to the existing costs, the savings associated with upgrades are even larger.

Parameters	As-is	Upgraded
Wire damping	No	Yes
Length, km	10	10
Spans	35	35
Dampers	0	400
Initial upgrade cost	0	US$8,000
Event	Cat III or above hurricane	
Spans damaged	6 (20%)	0
Replacement, m	1,800	0
Replacement cost	US$54,000	0

Table 6-19. Example of transmission line upgrade cost

6.8.4 Life-cycle cost

For energy infrastructure, life-cycle cost analysis (LCCA) is typically used in design. This topic will be further discussed in Chapter 8.

6.9 Discussion

In subject Caribbean countries, there are large energy (power) infrastructure including a large number of substations and many km of transmission lines. The courtiers are also located in one of the most hazard-prone areas in the world, experiencing a number of hurricanes annually and earthquakes periodically. These natural hazards have caused significant damage to the energy infrastructure in recent years causing economic distresses and delayed recovery. Accordingly, it is time-critical to plan and implement measures that results in a resilient energy infrastructure. Experience has shown that a given sub-class of energy infrastructure, there are a number of components that are the most susceptible to damage in natural hazard events. For these components, effective retrofitting and mitigation methods are available which reduce the expected damage to infrastructure and are cost effective. Table 6-20 presents a summary of findings related to the most critical components of energy infrastructure in the subject countries.

Subsector	Hazard	Vulnerable Component	Resilience Measure	Cost, % of initial cost (Est.)	
				Improvement Cost	Vulnerability Reduction
Substation components	Earthquake	Transformer	Improved anchorage	5%	20%
		Equipment	Qualified components	10%	40%

		Equipment	Spares	10%	--
		Support frames	Add passive dampers	5%	20%
Transmission system	Wind	Transmission tower	Strengthen steel members	20%	50%
		Transmission line	Add vibration dampers	5%	50%
Foundations	Flood, liquefaction		Add micro piles	40%	80%

Table 6-20. Summary of findings for power infrastructure in Caribbean

7. ADDITIONAL MEASURES

7.1 Introduction

In this chapter, additional measures that improve infrastructure resiliency are presented. These topics complement the improved design and strengthening measures for various infrastructure sectors discussed in the preceding chapters. When undertaking a country- or region-wide infrastructure program, it is crucial to begin with a project planning matrix and feasibility study that incorporate the measures discussed herein.

7.2 Construction quality management

A construction quality management program is a main component to improved design and has a substantial impact on the durability and resistance of infrastructure to natural hazards. A number of practices and procedures used elsewhere for large-scale retrofit projects could be applicable in the 16 subject countries to strengthen building, water, transport, and power infrastructure projects.

7.2.1 Construction database and record keeping

As part of a pre-hazard vulnerability assessment, which is discussed in the next section, it is important to compile a database of various structures. For example, in the transport sector, for each bridge, a bridge book can be prepared that includes key information, such as as-built information, inspection reports, and results of vulnerability assessments. The bridge book and database are updated as retrofitting proceeds. This will allow assessing the efficacy of retrofit measures after the next hazard and maintaining an updated database for each bridge[14].

In the U.S. State of California, Caltrans maintains a list of transportation infrastructure inventory; see Table 7-1. The state's management plan provides aggregates for the seismic vulnerability of bridges and any planning to address them; see Table 7-2.

Item	Quantity
Highway	9,500 km
State highway bridges	13,100
Local bridges	12,200

Table 7-1. California transportation infrastructure inventory (adapted from Caltrans, 2020)

Bridge	2019	2029
Total bridge deck,10^6 m^2	23.4	--
Potential seismic vulnerability,10^6 m^2	1.4	0.4
Percentage	6%	--

Table 7-2. Seismic retrofit management plan (adapted from Caltrans, 2020)

Examples of data maintained in a bridge book are presented in Table 7-3. A hypothetical example for power infrastructure is presented in Table 7-4. Note that as seismic or flood retrofitting progresses, entries are to be updated to indicate alterations. Similar data would need to be collected and archived for all classes of substation.

Structure name:	
<u>Construction information:</u>	
Year built	1960
Year retrofitted	1998
Length, m	48

[14] In the U.S., bridge inspection is mandatory on a biannual basis

Skew, degrees	20
No. of joints	2
No. of hinges	0
Spans	2@ 24
Railings	
Min. vertical clearance	
Width, m	8
No. of lanes	2
Speed, km/h	60
Structure description (original)	
RC deck on composite four (4) simply supported buildup steel plate girders on two-column RC column bents with cap and RC open end seat type abutments with monolithic wingwalls. Founded on driven RC piles.	
Structure description (Retrofits)	
Restrainer cables were placed at each bent on each girder. Steel casings were added to the columns. The footings were also widened which included the addition of new 0.6-m diameter driven steel piles	
Current condition and recommended action	
The river has eroded approximately 60 m of the eastern bank upstream. Rehabilitation involves placing rock around the pier and abutments at an estimated cost of $20,000 million to resolve the existing scouring problem	

Table 7-3. Example of data inventory in bridge book

Substation name:	
Construction information:	
Year built	1960
Year retrofitted	1998
Number of transformers	3 115 kv
Number of circuit breakers	6 68 kV
Number of elevated disconnect switches	1 3-phase 115 kv 1 3 phase 68 kV
Other components	0
Control building	URM retrofitted in 1998
Switch tower	Steel lattice
Structure description (original)	
URM control building and 115 kV equipment. One transformer was damaged during the 2013 hurricanes and flood and not repaired. For the two other units, enhanced anchorage plates were added in 1998. At that time, URM control building was also retrofitted with steel rods	
Structure description (Retrofits)	
Anchorage for two transformers added	
Current condition and recommended action	
Flood protection is needed for the control building. The damaged transformer would require replacement of repair. Currently, there are no spare components at the substation.	

Table 7-4. Example of data inventory in substation records

7.2.2 Electronic real-time monitoring

Effectiveness of seismic retrofitting is highly dependent on the quality of retrofit construction. Therefore, it is important to develop a construction inspection and record-keeping program. Examples of key construction activities as part of retrofitting of transport or building infrastructure include:

- Verifying construction complies with structural drawings
- Inspecting materials, such as steel and reinforcement, at delivery and collecting mill certification sheets

- Sampling of concrete and reinforcement
- Maintaining records of material testing data
- Monitoring pile driving
- Checking concrete mix design and delivery time
- Checking size, location, and spacing of reinforcement
- Verifying calculations for any falsework and excavation, and shoring
- Checking shop drawings for steel and prestressing
- Conducting or attending safety meetings
- Electronically recording daily activities in a log

An example of a daily activity log is presented in Table 7-5 and Table 7-6 for transport and power infrastructure retrofit projects, respectively.

Bridge no.						RPT #		**710**						
State						DATE		**2022_11_07**						
District						Day		M	T	W	T	F	S	S
Route						SHIFT		START		730	STOP		1600	
Br	No	Location	Activity											
W	1	Pier 3	Drive steel piles											
E	2	Pier 4	Excavate around existing footing											
W	3	Pier 5	Prepare column for placement of FRP casing											
W	4	Deck	Strip and cleanup; move panels to staging area											
W	5	Falsework	Prep for removal of 30-m beams											
		Activity			1	2	3	4	5	6	7	8	Σ	Abs
		Employee	Job	Equip.										
Main contractor			Super											
	1		Foreman											
	2		Carpenter											
	3		Pile driver											
	4		Laborer											
	5		Laborer											
	6		Operator											
	7		Oiler											
	8		Apprentice											
Sub	1		Foreman											
	2		Iron worker											
	3		Iron worker											

Table 7-5. Example of construction inspector daily activity log for transport construction or retrofit

Substation no.						RPT #	**710**						
State						DATE	**2022_11_07**						
District						Day	M	T	W	T	F	S	S
Route						SHIFT	START		730	STOP		1600	
Subs	No	Location	Activity										
	1	Transformer 1	Remove existing bolts										
	2	Transformer 2	Finish installation of bracket anchorage										
	3	Breaker	Wire ropes were added to the base										
	4	Switch frame	Clamped steel tubes to the frame legs										
	5	Control room	Bracing locations for cable trays were marked										

		Activity			1	2	3	4	5	6	7	8	Σ	Abs
		Employee	Job	Equip.										
Main contractor			Super											
	1		Foreman											
	2		Carpenter											
	3		Steel worker											
	4		Laborer											
	5		Laborer											
	6		Welder											
	7		Concrete mason											
	8		Apprentice											
Sub	1		Foreman											
	2		Electrician											
	3		Electrician											

Table 7-6. Example of construction inspector daily activity log for power construction or retrofit

7.2.3 Use of proven technologies

Using proven technologies reduces the recovery time and increases significantly the quality of work. Bailey bridges are steel truss bridges that can be constructed quickly. These bridges were developed during WWII as a rapid replacement for damaged structures. Key advantages of this bridge type include:

- Rapid installation
- Bridge is launched and thus minimal equipment is needed. In some cases, human power alone can be used to launch these bridges
- The bridge is light and strong
- Although primarily used as a temporary structure, they can be left in place for long-term use because bridges are designed per AASHTO LRFD code, can have multiple lanes, and incorporate barriers.
- They are inexpensive
- The bridge can span 80 m or more and thus in many cases eliminate the need for foundations when crossing rivers. This in-turn mitigates flooding hazard and foundation scour.
- Since the bridge is assembled in identical segments (see Figure 7-1), the construction quality is improved, as the construction crew will be doing repetitive work.
- It is an ideal candidate for use in the aftermath of natural disasters that have damaged or washed away existing bridges. In a number of subject countries, this type of bridge has been used in recent years for this specific purpose; see Figure 7-2.

Figure 7-1. Example of Bailey (steel truss) bridge

Trinidad and Tobago

Trinidad and Tobago

Saint Vincent and the Grenadines

Dominica

Haiti

Barbados

Jamaica

Saint Lucia

Figure 7-2. Application of temporary Bailey bridges in subject countries (various sources)

Although Bailey bridges can provide a reliable means of transportation when properly constructed, it is important to ensure compliance with manufacturer requirements. For example, bridges need to be assembled properly and constructed using preapproved parts for which the design was based on. Additionally, these are narrower bridges and there are vehicular axle weight restrictions and speed limitations. Failure to comply with all the construction and use requirements can cause damage or collapse of these bridges. Examples in Figure 7-3 show failures due to overload (India), heavy rains and ensuing flooding (likely due to inadequate foundation) (Dominica), and lack of proper inspection during construction (Trinidad and Tobago).

Bridge collapse, Uttarakhand India (INDIA TV, 2020)

Bridge failure, Dominica (Dominica News, 2017)

Bridge collapse, Trinidad and Tobago (Jyoti Communication, 2010)

Figure 7-3. Collapse of Bailey bridges due to overload or poor construction

7.2.4 Contract incentives

As part of record keeping for new construction or retrofit works, it is important to maintain a daily record of activities, as discussed previously. In addition, records of contractor quality of work for each project can be kept. This is then input into a database and can be used as part of award process for future infrastructure rehabilitation or reconstruction projects. A successful example of such an approach is the building infrastructure construction in Hong Kong, where the Hong Kong Housing Authority (HA) has been developing construction quality management systems and tools since the 1990s. In 1990, the development of the Performance Assessment Scoring System (PASS) was initiated, and this was reviewed periodically and updated in 2001.

PASS is an indicator that serves as a tool to measure the quality of work. The project team and PASS assessment team conduct site inspections, desktop reviews, and record checks. The PASS items related to construction quality of structural work can be grouped as follows:

Category	Item
Engineering	Reinforcement
	Formwork and falsework
	Finished concrete
	Construction records

Site formation	Excavation
	Compaction
Architectural	Floors
	Walls
General	Management
	Documentation
	Progress assessment
	Milestone dates for structural work
Environment	Storage of material
Safety	Safety program and record

Table 7-7 Components of PASS (adapted from HA, 2020)

All building construction sites undergo monthly PASS inspections. Every three months, a quarterly PASS composite score for each individual contractor's projects is completed. The following metrics are developed:

- Contractors' League: considers the overall performance of contractors across all projects being undertaken
- Composite Target Quality Score is in the upper quartile (25 percentile) of the league.
- Composite Lower Score Threshold is in the lower quartile (75 percentile) of the league.

Composite Target Quality Score and Composite Lower Score Threshold are used to assess the overall trend of construction quality over time.

HA has adopted a preferential tender award system where preference is given to better-performing contractors by evaluating both the tender price and performance potential. The formula used for tender evaluation is determined by calculating the score of all fully capable tenders and selecting one with the highest score from the following formula:

$$60 * \frac{Lowest\ price\ among\ tenders}{Tender\ price} + 40 * \frac{Tender\ performance\ score}{Highest\ performance\ score\ of\ tenders}$$

7.3 Screening and prioritization

Power, water, transport, and building structures are key lifeline infrastructure assets for which vulnerability to natural hazards needs to be identified and mitigated. A critical component is the condition assessments for vulnerability.

Prior to undertaking a retrofitting program, it is crucial to undertake a comprehensive risk assessment program to identify vulnerable infrastructure and components for these infrastructures that require retrofitting. This involves site visits, preparing or reviewing as-built drawings, and performing, at the very least, preliminary analyses to confirm the said vulnerabilities.

A screening form for infrastructure needs to be developed prior to the field work. Example forms for transport and power infrastructure are presented in Table 7-8 and Table 7-9, respectively. A database of the completed screening forms can then be created. This database can be used to program the infrastructure for strengthening and prioritize the structures based on monetary resources. The database should then be updated after the successful completion of retrofitting projects.

Bridge Screening Form (Earthquake, wind, and flood)							
Description of bridge							
Inspector		Date		Longitude		Latitude	
Br. No.		Bridge name		District		Location	
Structural assessment							

	Description		Weight	Score
	Abutment seat <300 mm			
	Columns, poorly confined			
	Timber piles			
	Steel girder superstructure, depth >1.2 m			
	Skew ≥30 degrees			
	Simple span precast bridge			
	Gap between abutment backwall and bridge >150 mm			
	No top mat footing reinforcement			
	Fatigue vulnerable detail for steel connections			
	Exposed foundation			
	Evidence of local scour and no riverbank protection			
Structural score				
Hazard assessment				
	PGA the site of 1000-year event			
	Soil factor (very competent to highly liquefiable)			
	Design wind speed for site			
	Pier cap below 100-year flood elevation			
	River susceptible to flooding and riverbank has eroded			
Hazard score				
Importance assessment				
	Average daily traffic (ADT)			
	Detour length			
Importance score				
Overall score				
Additional comments				

Table 7-8. Example of bridge screening form

Substation Screening Form (Earthquake, wind, and flood)								
Description of Substation								
Inspector		Date		Longitude		Latitude		
Capacity, kV		Substation name		District		Location		
Structural assessment								
	Description					Weight	Score	
	Unanchored transformers							
	Unanchored equipment							
	HV equipment does not have seismic plate							
	Support structures designed for seismic forces							
	Control building has no flood protection							
	Transmission tower not designed for code level wind loading							
	Transmission wires have no Stockbridge damping							
Structural score								
Hazard assessment								

	PGA the site of 1000-year event			
	Soil factor (very competent to highly liquefiable)			
	Design wind speed for site			
	Construction below 100-year flood elevation			
Hazard score				
Importance assessment				
	Average people served, daily			
	Redundant substation can provide electricity			
Importance score				
Overall score				
Additional comments				

Table 7-9. Example of substation screening form

7.4 Preventive maintenance

The long-term health of infrastructure depends on regular maintenance and inspection. As part of feasibility studies, the stakeholders should consider all three phases in the service life of infrastructure, namely: design, construction, and maintenance.

For example, in the transport sector, for paved roads, AASHTO (2012) requires the following:

- Assess pavement condition over time
- Identify pavement preservation recommendations
- Evaluate the long-term effects of changes in material properties, design, and construction

As part of the program, data is collected regularly (typically biennially in the U.S.). Data includes the following:

- General information, such as road locations, number of lanes, lane width, shoulder width, pavement type, shoulder type
- Layer type, layer thickness, layer strength, subgrade type, drainage information, environmental data, material properties
- Traffic data
- Pavement condition

This data is then organized in a database to warehouse data that can be used to identify key parameters impacting the pavement performance and assess the factors that lead to more robust pavements during natural disasters, such as flooding. Data can also be used to develop recommendations for pavement treatment. Date is then used to assess the efficacy of preventive maintenance. As seen in Figure 7-4, preventive measures lead to extended service life. The cost associated with such measures is significantly less than replacement – which is more likely absent preventive maintenance – cost after natural disasters.

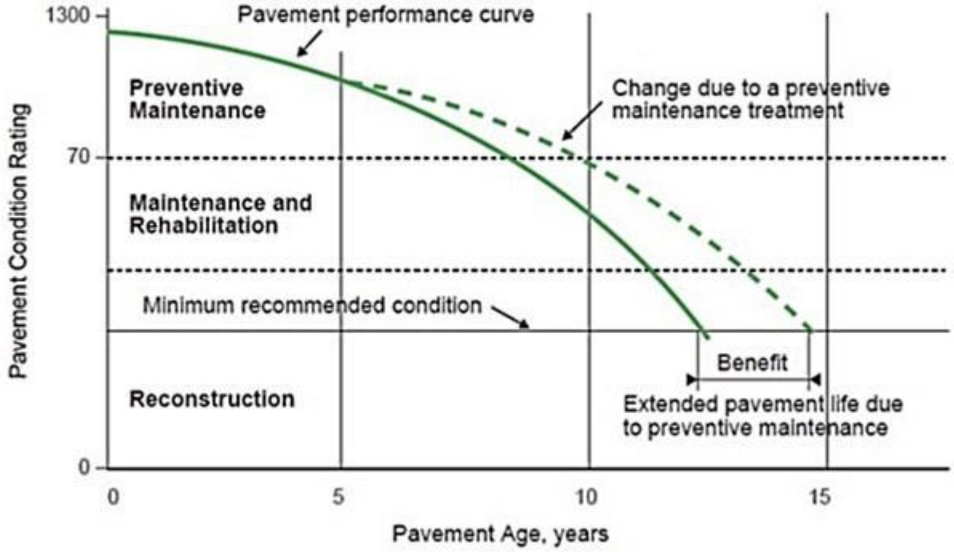
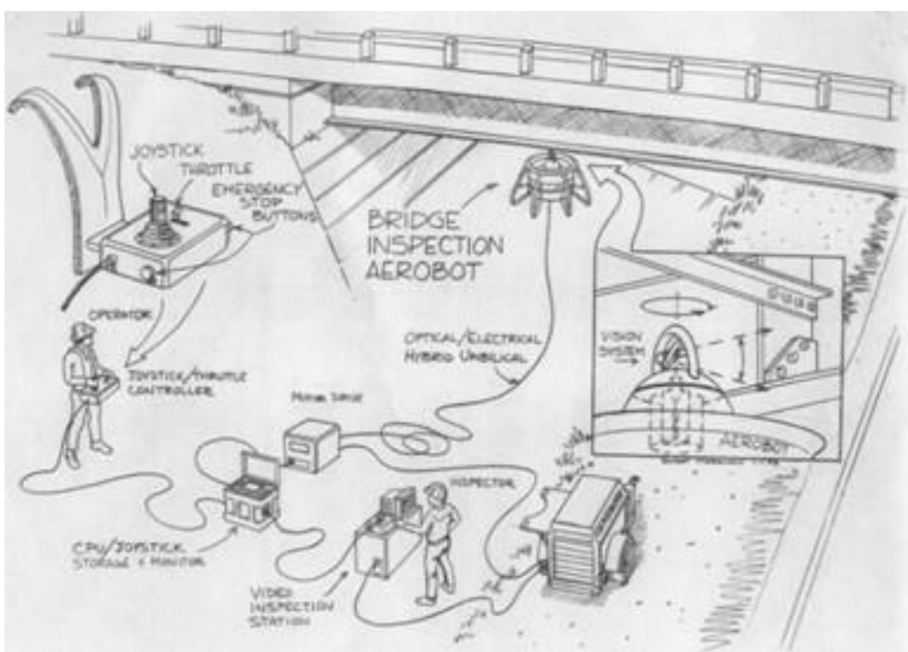


Figure 7-4. Pavement management systems flowchart (AASHTO, 2012)

7.5 Use of advanced technologies for inspection and damage assessment

In many of the subject countries, there is a shortage of skilled workers able to conduct detailed regular inspections of transport infrastructure or to perform rapid assessment in the aftermath of natural disasters. New technologies, such as satellite imagery, drones, and light detection and ranging (LiDAR), can be used to perform such tasks quickly, accurately, and can allow for transmission of data digitally in real time.

For example, in California, drones are being used as a key component by the transportation department in the biannual structural inspection of transportation infrastructure; see Figure 7-5 (Caltrans, 2020). Unmanned aerial vehicles (UAV or drones) provide the flexibility to inspect bridges and reach hard-to-access areas, collect photographs and videos, take measurements, and document anomalies. The inspection data is then uploaded in real time for review and processing by engineers.



Schematics

Drone used in inspection

Performing inspection

Figure 7-5. Drone inspection of transportation infrastructure (Caltrans, 2020)

Similarly, drones can be used in the aftermath of natural disasters to perform damage assessment reconnaissance. This allows inspection of hard-to-access places. Additionally, since the structures (including inside damaged units) can be remotely inspected, this approach eliminates the need to be placed in unsafe environments to the inspectors, who would otherwise have to visit unsafe structures for data collection. Examples are presented in Figure 7-6.

2014 Napa Earthquake (CA)

2017 Hurricane Irma (St. Martin)

Figure 7-6. Post-event damage assessment by drones (various)

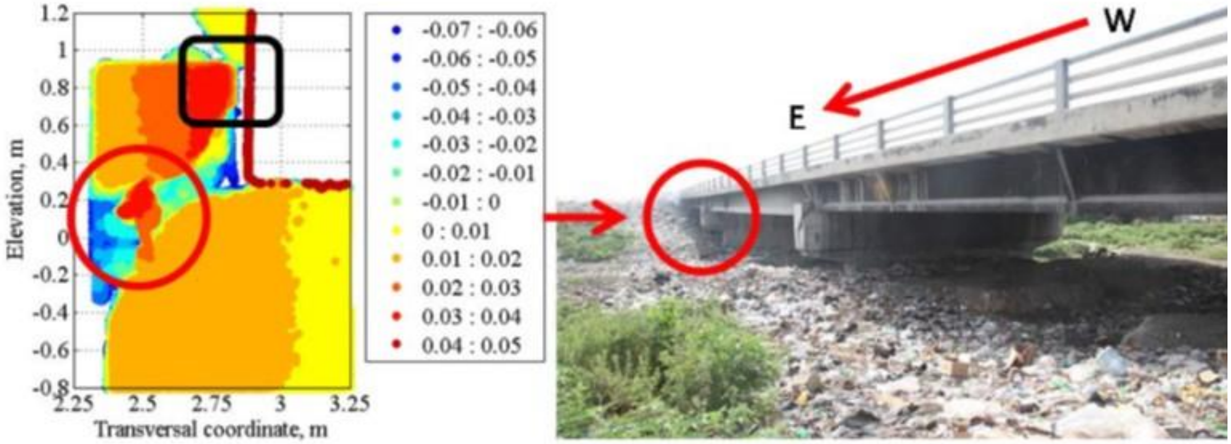
Light detection and ranging (LiDAR) and Global Positioning Systems (GPS) can be used to monitor landslides and provide warnings in the event of land movement above a certain threshold. Beginning in 2006 and following heavy rainstorms, the 800,000-m^3 Ferguson Slide in California blocked the roadway and resulted in significant economic damage. The landslide has been monitored remotely and will be monitored during the construction of a new rock shed. LiDAR and GPS data are collected as part of a system to monitor the displacement, velocity, and acceleration of the slide and to provide warnings; see Figure 7-7.

This technology was successfully used in Haiti in the aftermath of the 2010 earthquake. For example, a bridge close to the epicenter was inspected manually and it was also scanned. The damage assessment collected by engineers compared favorably with damage detected during the scan; see Figure 7-8 (Takhirov and Mosalam, 2014).

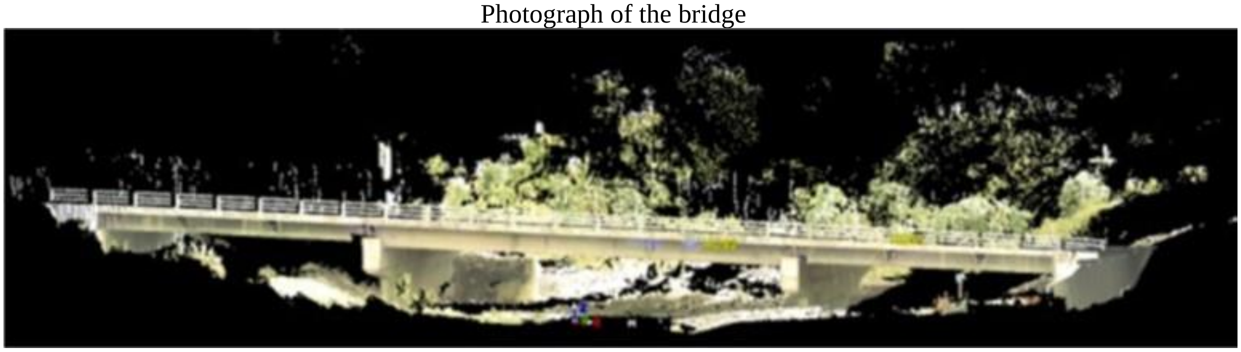
Landslide blocking the roadway

Monitoring the slide

Figure 7-7. Monitoring of Ferguson Landslide (Caltrans, 2020)



Photograph of the bridge



Scan of the bridge

Figure 7-8. Laser scanning of bridge in the aftermath of 2010 earthquake (Takhirov and Mosalam, 2014)

7.6 Monitoring and early warning system (EWS)

7.6.1 United States applications

In the U.S., Japan, and elsewhere, networks of strong motion sensors are used to collect earthquake shaking data, determine the expected level of damage, and predict infrastructure response. For example, important bridges in California have been instrumented; see Figure 7-9.

In the U.S., the National Hurricane Center (NOAA, 2020) provides key information for tracking and monitoring of hurricanes and provides guidance and advisories. The center also tracks hurricanes in the Caribbean region. Figure 7-10 presents an example of hurricane tracking for Hurricane Irma in the Caribbean region.

In California, rainfall is monitored using a system of water gauges and uses radar and satellite information for rainfall projections; see Figure 7-11 (USGS, 2005). Thresholds are set for the incipient flood rainfall and when the thresholds are exceeded, warnings are sent to responsible emergency agencies. A similar methodology can be implemented in the Caribbean to alert the infrastructure managers (for example, transportation officials) of danger of imminent flood. This warning will alert them to close roads and bridges. A reduction in traffic loading on paved roads reduces the severity of structural damage to this infrastructure, because a combination of traffic and flood load is more impactful on paved roads than flood alone.

Figure 7-9. Instrumented bridges (Caltrans, 2020)

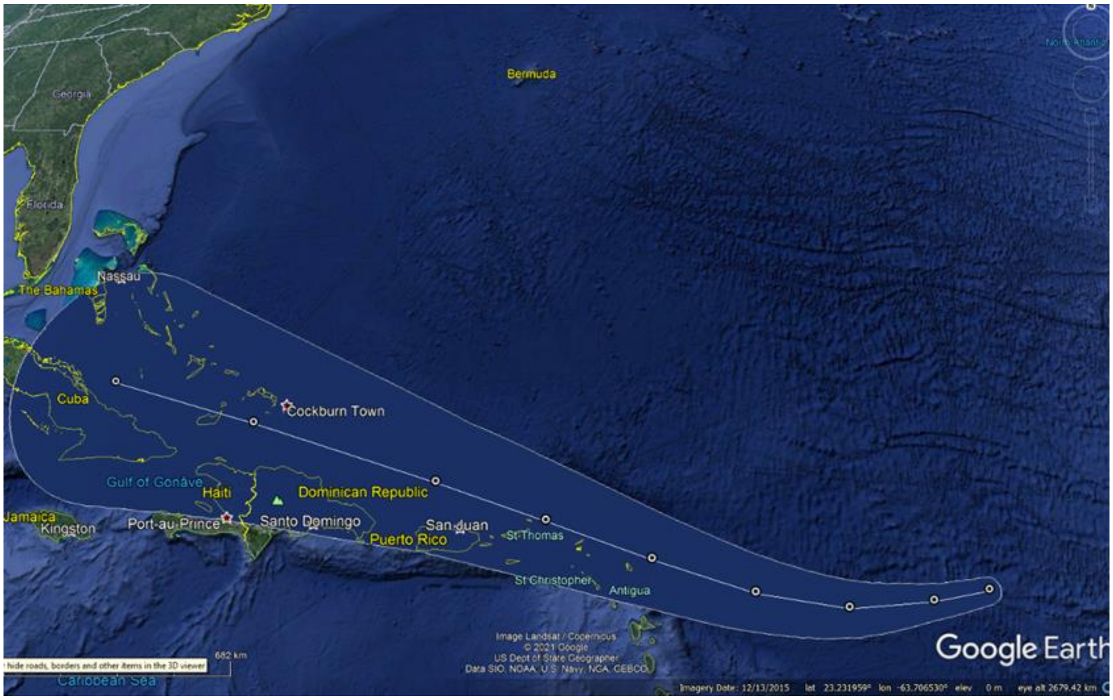


Figure 7-10. Hurricane Irma in the Caribbean (NOAA, 2020)

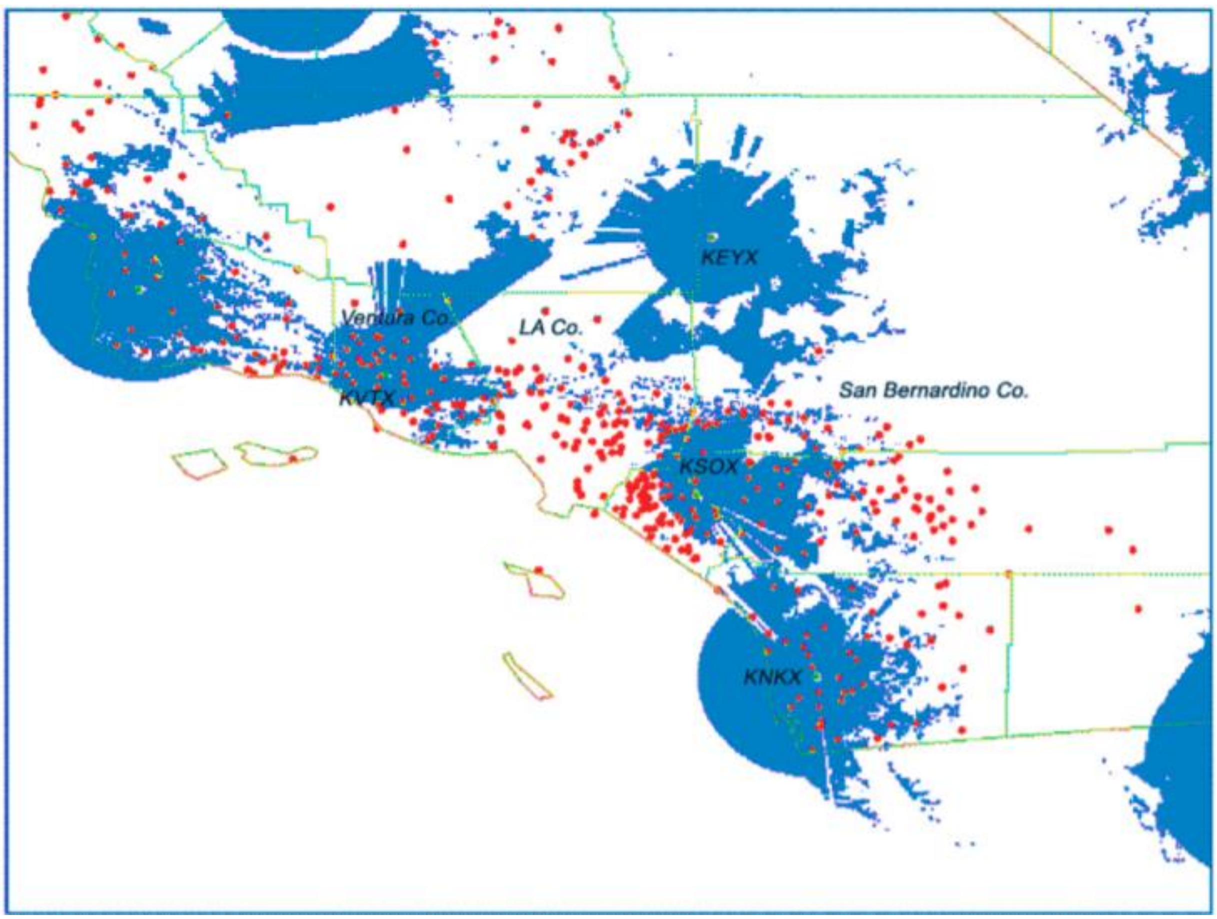


Figure 7-11. Area of radar coverage (blue regions) and rain gauge locations (red dots) in Southern California (USGS, 2005)

7.6.2 Application to the Caribbean

The University of the West Indies Seismic Research Centre (UWI SRC) operates a network of strong motion stations in the region; see Figure 7-12. Data from the networks can be used to expedite response and collect critical information related to earthquake shaking and infrastructure response.

The Caribbean Institute for Meteorology and Hydrology (CIMH) provides weather forecasts, including for wind speed and rainfall, for the subject countries. CIMH uses statistical techniques and computer modeling to forecast hurricanes and provides forecasts in 6-hour intervals extending to 72 hours. Figure 7-13 presents an example of a previous hurricane forecast available from CIMH.

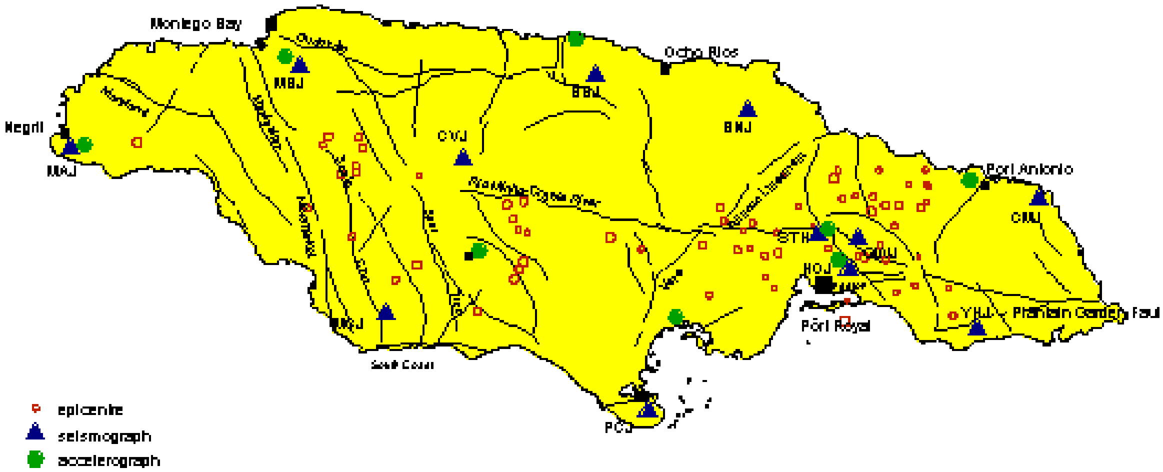
In a number of the subject countries, rain gauges are used to establish thresholds and provide early warnings for incipient flooding; see Figure 7-14. However, there are two issues for application of this approach to a number of smaller island subject countries that have a volcanic geology, namely: i) response time is limited; and ii) there is limited availability of skilled staff for monitoring. To solve this shortcoming, CIMH is planning to implement a flash flood guidance system based on available flood models, remote satellite data, and in-situ instrumentation. A schematic of the system is presented in Figure 7-15.

Currently, the governments of individual subject countries are responsible for issuing the early warning for natural disasters. This effort in-turn is supported by UWI SRC, CIMH, and the Caribbean Disaster Emergency Management Agency (CDEMA). However, the lack of coordination among agencies and delineation of responsibilities varies among the countries. For example, although hydromet agencies are operating continuously, they do not have the legal mandate to issue warnings. To improve the current conditions, CDEMA has initiated an early warning systems (EWS) project with the following objectives (CDEMA, 2020):

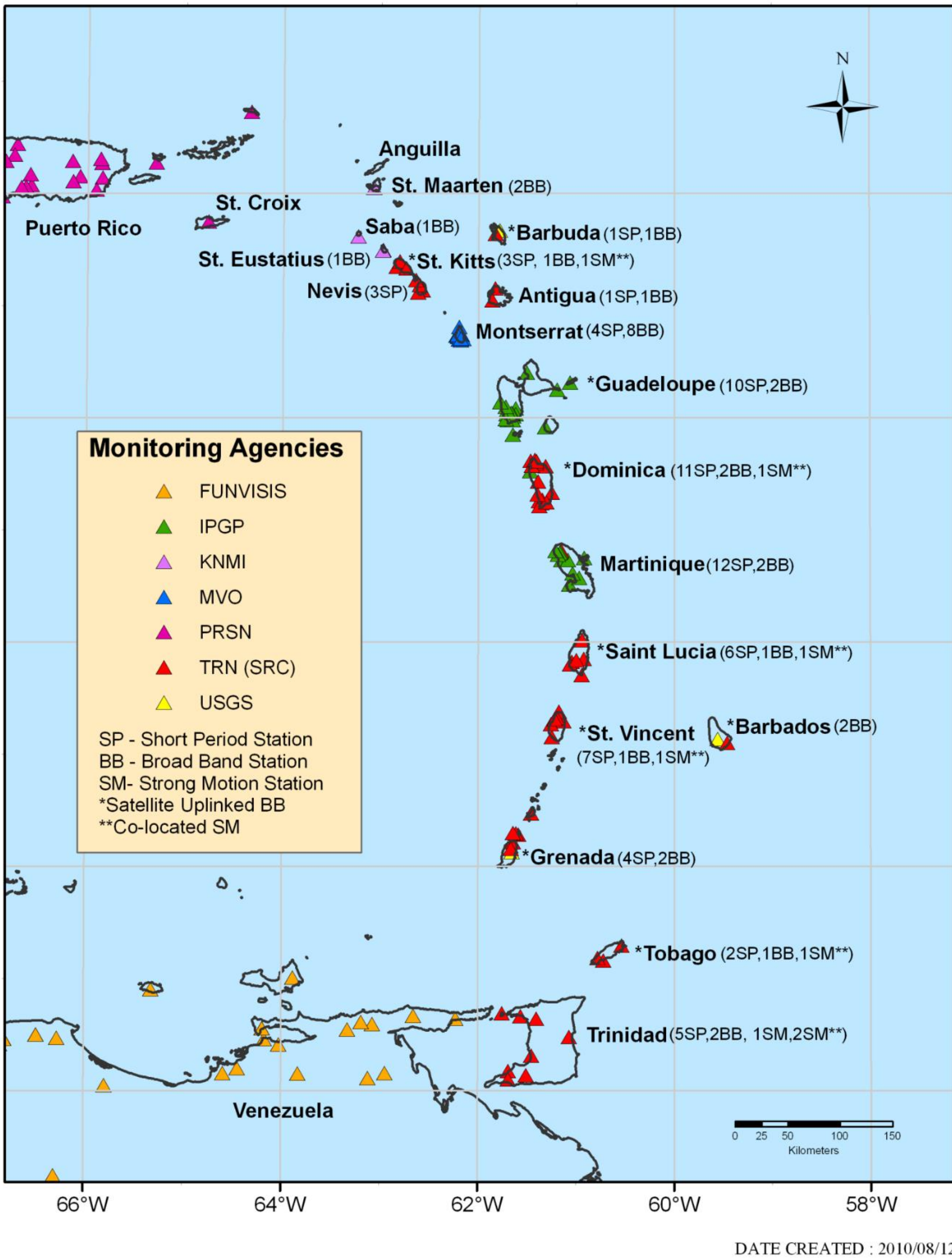
> *Strengthening integrated early warning systems for more effective disaster risk reduction in the Caribbean through knowledge and the transfer of tools.*
>
> *Strengthen integrated and cohesive preparedness capacity at a regional, national and community level in the Caribbean.*

The checklist developed by CDEMA is presented in Table 7-10. Note that one of the key action items is to integrate the EWS among various countries. CDEMA (2020) states:

> *This project, which is being implemented in Antigua and Barbuda, Dominica, the Dominican Republic, Haiti, Saint Lucia, Saint Vincent and the Grenadines and Cuba over an 18-month period, seeks to strengthen disaster preparedness and risk reduction through Integrated Early Warning Systems (EWS). Working in this important element for disaster risk reduction, is expected to enhance the prevention, mitigation and response capacities at both the institutional and community level, based on mutual learning and collaboration between countries and regional institutions working in disaster risk reduction across the Caribbean.*



Jamaica (8 stations)



Eastern Caribbean

Figure 7-12. Seismic strong motion stations in subject countries (UWI, 2020)

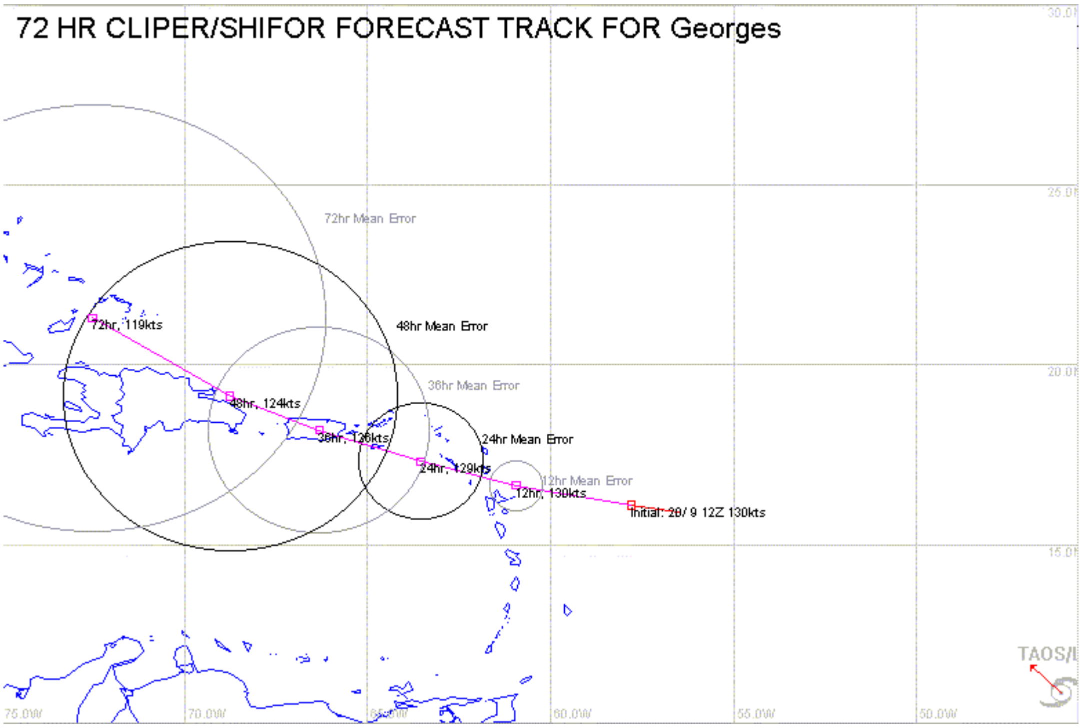


Figure 7-13. Forecast track for the 1998 Hurricane Georges (CIMH, 2020)

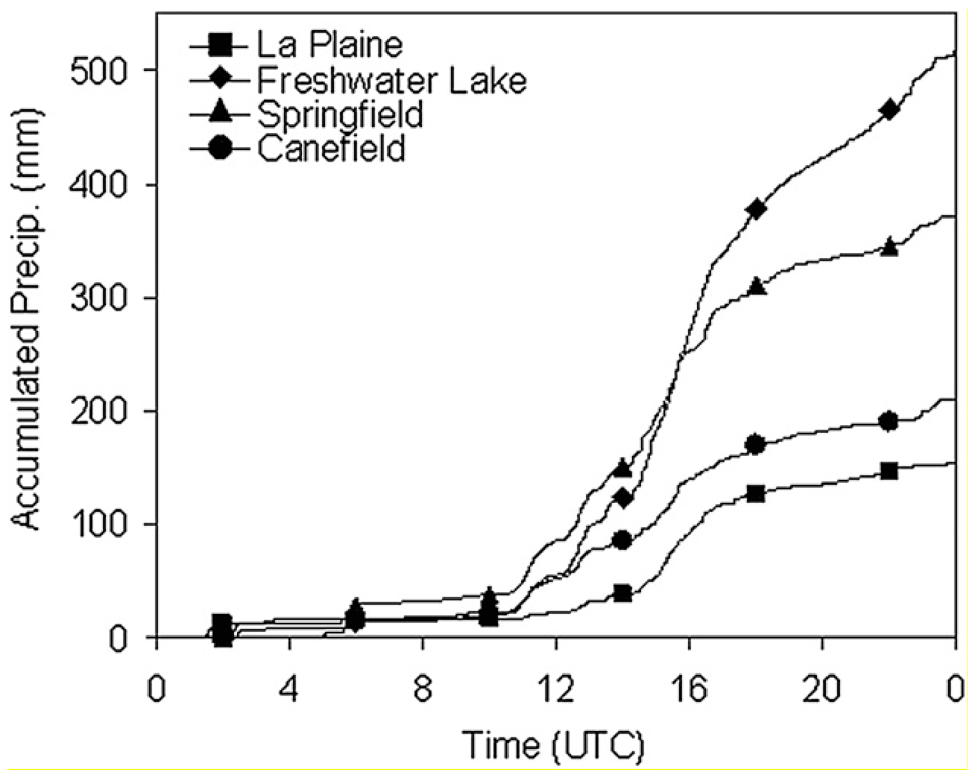


Figure 7-14. Recorded rain gauge data in Dominica during Hurricane Dean (Smith et al., 2008)

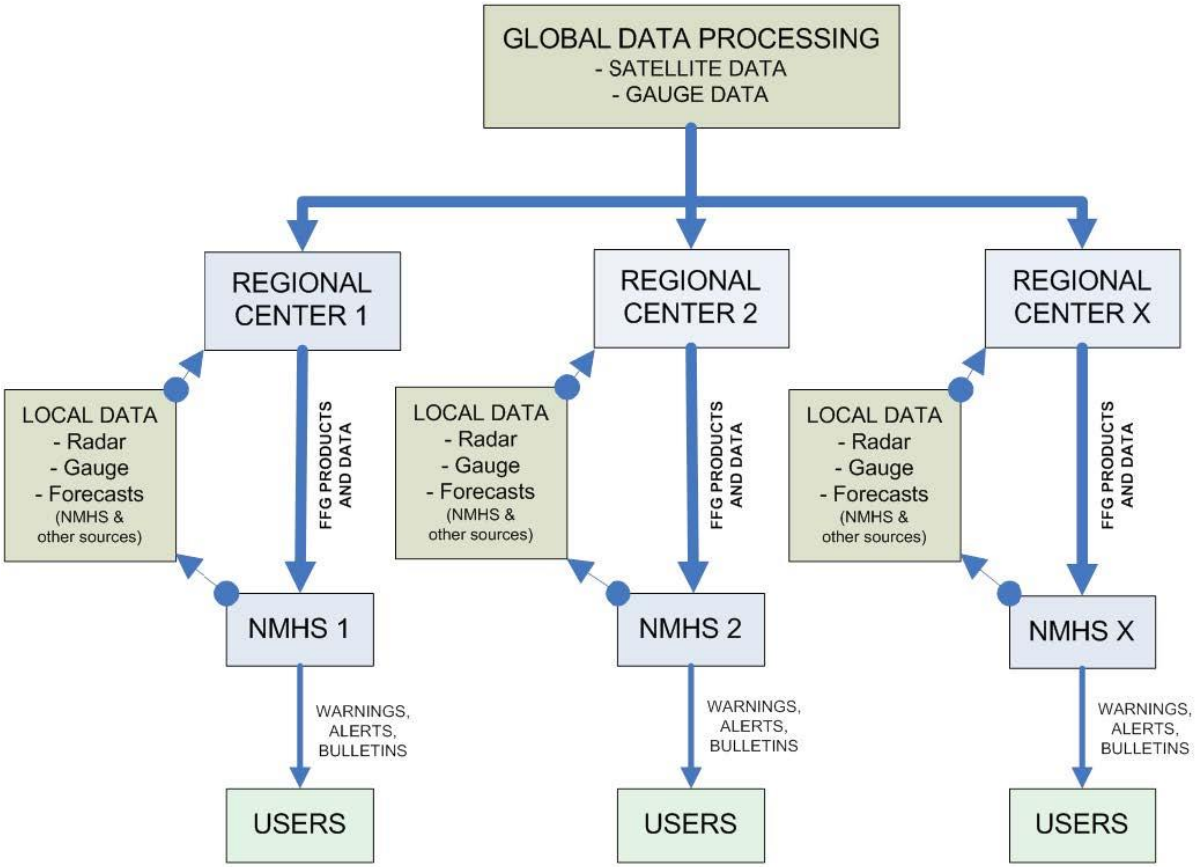


Figure 7-15. Schematics of flash flood guidance (HDRFFG, 2016)

Actions (partial list shown)
Monitoring network established that monitors hazards that impact your country
Measurement parameters and specifications documented for each relevant hazard
Technical equipment, suited to local conditions, circumstances and gender differentiated needs, in place and personnel trained in its use and maintenance
Monitoring data received, processed and available in an interoperable format in real time, or near-real time
Monitoring data and metadata routinely curated with quality controls, archived and accessible for verification, research purposes and other applications
Monitoring hardware and software maintenance is addressed routinely and costs and resources are considered from the beginning to ensure the optimal operation of the system over time
The system is able to combine and benefit from new and older technology allowing for exchange of data among countries with different technical capabilities
Data analysis and processing, modelling, prediction and warning products generation based on accepted scientific and technical methodologies and disseminated within international standards and protocols.
New data analysis and processing, modelling, prediction and warning products can be integrated easily in the system as science and technology evolve.
Warning centres are operational at all times (24 hours/day, seven days/week) and staffed by trained personnel, following appropriate national and international standards.
Warnings generated and disseminated in an efficient, timely manner for each type of hazard.
Warning system(s) subjected to regular system-wide tests and exercises

Table 7-10. Portion of EWS (adapted from CDEMA, 2020)

7.7 Response protocol

7.7.1 International best practice

A post-event inspection (damage assessment) program needs to be implemented by regional groups or national governments to rapidly respond in the event of a natural disaster impacting infrastructure. As part of this program, an inspection form needs to be used; an example is shown in Figure 7-16 and Figure 7-17 for building structures and for earthquakes or hurricane and flood, respectively. These forms can be updated to tailor to construction typologies in the Caribbean and similar forms for other infrastructure can be developed. Training and capacity building for government engineers and consultants needs to be undertaken in order to certify sufficient numbers of qualified engineers to respond after an event. Refresher online training can be conducted every several years to ensure trainees are up to date with any changes. A database of certified individuals should be maintained, including contact information to allow timely response. Because in the aftermath of a natural hazard event, unsafe conditions could be present, safety training should be incorporated as part of the overall program and safety information should be included in the booklet.

ATC-20 Rapid Evaluation Safety Assessment Form

Inspection

Inspector ID: ______ Inspection date and time: ______ ☐ AM ☐ PM

Affiliation: ______ Areas inspected: ☐ Exterior only ☐ Exterior and interior

Building Description

Building name: ______

Address: ______

______

Building contact/phone: ______

Number of stories above ground: ____ below ground: ____

Approx. "Footprint area" (square feet): ______

Number of residential units: ______

Number of residential units not habitable: ______

Type of Construction

☐ Wood frame ☐ Concrete shear wall

☐ Steel frame ☐ Unreinforced masonry

☐ Tilt-up concrete ☐ Reinforced masonry

☐ Concrete frame ☐ Other: ______

Primary Occupancy

☐ Dwelling ☐ Commercial ☐ Government

☐ Other residential ☐ Offices ☐ Historic

☐ Public assembly ☐ Industrial ☐ School

☐ Emergency services ☐ Other: ______

Evaluation

Investigate the building for the conditions below and check the appropriate column.

Observed Conditions:	Minor/None	Moderate	Severe
Collapse, partial collapse, or building off foundation	☐	☐	☐
Building or story leaning	☐	☐	☐
Racking damage to walls, other structural damage	☐	☐	☐
Chimney, parapet, or other falling hazard	☐	☐	☐
Ground slope movement or cracking	☐	☐	☐
Other (specify) ______	☐	☐	☐

Estimated Building Damage (excluding contents)

☐ None
☐ 0–1%
☐ 1–10%
☐ 10–30%
☐ 30–60%
☐ 60–100%
☐ 100%

Comments: ______

Posting

Choose a posting based on the evaluation and team judgment. *Severe* conditions endangering the overall building are grounds for an Unsafe posting. Localized *Severe* and overall *Moderate* conditions may allow a Restricted Use posting. Post INSPECTED placard at main entrance. Post RESTRICTED USE and UNSAFE placards at all entrances.

☐ **INSPECTED** (Green placard) ☐ **RESTRICTED USE** (Yellow placard) ☐ **UNSAFE** (Red placard)

Record any use and entry restrictions exactly as written on placard: ______

Further Actions Check the boxes below only if further actions are needed.

☐ Barricades needed in the following areas: ______

☐ Detailed Evaluation recommended: ☐ Structural ☐ Geotechnical ☐ Other: ______

☐ Other recommendations: ______

Comments: ______

Figure 7-16. Post-earthquake damage assessment form (ATC, 2020)

ATC-45 Rapid Evaluation Safety Assessment Form

Inspection

Inspector ID: ______________________ Inspection date: ______________________

Affiliation: ______________________ Inspection time: ______________ ☐ AM ☐ PM

Areas inspected: ☐ Exterior only ☐ Exterior and interior

Building Description

Building name: ______________________

Address: ______________________

____

Building contact/phone: ______________________

Number of stories: ______________________

"Footprint area" (square feet): ______________________

Number of residential units: ______________________

Type of Building

☐ Mid-rise or high-rise ☐ Pre-fabricated

☐ Low-rise multi-family ☐ One- or two-family dwelling

☐ Low-rise commercial

Primary Occupancy

☐ Dwelling ☐ Commercial ☐ Government

☐ Other residential ☐ Offices ☐ Historic

☐ Public assembly ☐ Industrial ☐ School

☐ Emergency services ☐ Other: ______________

Evaluation

Investigate the building for the conditions below and check the appropriate column.

Observed Conditions:	Minor/None	Moderate	Severe
Collapse, partial collapse, or building off foundation	☐	☐	☐
Building significantly out of plumb or in danger	☐	☐	☐
Damage to primary structural members, racking of walls	☐	☐	☐
Falling hazard due to nonstructural damage	☐	☐	☐
Geotechnical hazard, scour, erosion, slope failure, etc.	☐	☐	☐
Electrical lines / fixtures submerged / leaning trees	☐	☐	☐
Other (specify) ______________	☐	☐	☐

☐ See back of form for further comments.

Estimated Building Damage
(excluding contents)

☐ None

☐ >0 to <1%

☐ 1 to <10%

☐ 10 to <30%

☐ 30 to <70%

☐ 70 to <100%

☐ 100%

Posting

Choose a posting based on the evaluation and team judgment. Severe conditions endangering the overall building are grounds for an Unsafe posting. Localized Severe and overall Moderate conditions may allow a Restricted Use posting.

☐ **INSPECTED** (Green placard) ☐ **RESTRICTED USE** (Yellow placard) ☐ **UNSAFE** (Red placard)

Record any use and entry restrictions exactly as written on placard: ______________________

____

____

Number of residential units vacated: ______________________

Further Actions Check the boxes below only if further actions are needed.

☐ Barricades needed in the following areas: ______________________

____

☐ Detailed Evaluation recommended: ☐ Structural ☐ Geotechnical ☐ Other: ______________

☐ Substantial Damage determination recommended

☐ Other recommendations: ______________________

____

☐ See back of form for further comments.

Figure 7-17. Post-hurricane and flood damage assessment form (ATC, 2020)

7.7.2 Applications to the Caribbean

In recent years, both electronic and mobile-based assessment forms have been developed. Additionally, although these forms have been developed for use in the U.S., modified versions have been developed to account for local construction practices and are used in a number of countries. For example, a version tailored for Haiti was used in the aftermath of the 2010 Earthquake in Haiti (Miyamoto and Gilani, 2012).

Currently, most governments dispatch engineers to perform post-event assessments. However, there is no formal training, uniform documentation, or standardized program. Organizations such as CDEMA can undertake development of a standardized program across the region comprising the following:

- Preparation of paper and digital forms for data collection
- Mechanism for storing and accessing collected data
- Providing training and certifications to inspectors
- Maintaining a list and contact information of inspectors for rapid development after major events
- Implementing a safety protocol for inspection

7.8 Regional approach

A number of measures proposed in this chapter rely on the subject countries having both technical capacity and financial resources to implement such actions. While this may be the case for counties such as Trinidad and Tobago, Jamaica, and Barbados, implementation would be challenging for smaller countries due to a lack of funds or availability of qualified personnel. This highlights the need to undertake a region-wide approach to implementing such measures and other actions that can enhance infrastructure resiliency in the Caribbean. Current examples of such coordination include:

- CUBIC building code (see Chapter 3)
- Seismic strong motion instrumentation and hurricane monitoring by SRC and CIMH (see Section 7.6)
- Development of EWS by CDEMA (see Section 7.6)
- Regional climate change adaptation and disaster risk management by the Caribbean Catastrophe Risk Insurance Facility (CCRIF SPC, 2020) that provides financial resources to limit the immediate impact of natural disasters to (19) member countries
- Caribbean Building Inspector Training Courses (CDMP, 1999) which involve training of building inspectors at the University of Technology in Jamaica

However, the regional organizations responsible for these efforts are dependent on non-regional funding for these programs, and they are frequently not sustained after the funding expires. Examples of low-cost collaborative efforts that can be implemented regionally include the following, and these steps would provide components for development of a regional infrastructure program:

- Regional and departmental levels plan regular and periodic communication for development of a resilient planning platform.
- Coordination for adoption of design codes
- Implementation of a uniform material standards for construction
- Open-source sharing of data from natural disasters, including structural damage data and recording hazard intensities
- Provisions for uniform minimum design and engineering standard of practice and care across the region
- Documentation and sharing of cost for construction of infrastructure
- Recoding of costs and associated downtime associated with various infrastructure damage as a consequence of natural hazards

7.9 Discussion

As part of project planning and feasibility studies for retrofitting of key infrastructure assets, it is critical that the infrastructure owners and managers consider the entire service life of the structures, including design, construction, and maintenance. While the primary focus of the report has been on improved design practices, both construction and maintenance play a critical role in developing and sustaining a resilient community and network of infrastructure. Improvements in construction quality management and regular preventive maintenance are both critical. Additional measures, such as development of a vulnerability database, instrumentation, including early warning systems, and response protocols, including rapid post-event assessments, are complementary actions that can enhance the likelihood of a successful long-term project and reduce human and economic losses. Further, it will protect infrastructure in the event of natural disasters.

8. MITIGATION COST AND FINANCING

8.1 Introduction

Infrastructure planners face competing economic constraints. These projects are typically not inexpensive. Additionally, given the scope and complexity, there is inherent risk (for example, a change in scope, delays in meeting a schedule milestone, or increased cost) associated with infrastructure improvement projects. To implement strengthening techniques, it is critical to conduct a feasibility study (FS). As part of this study, the focus should be on life-cycle cost (LCC) and not on initial cost, because infrastructure are typically in service for decades and accumulated upkeep and repair costs over service life can be significantly more for an option with a lower initial cost. To assist with financing and project management, FS could incorporate and examine innovative contracting options that allow the owners to share some of the project risks with qualified contractors. These topics are discussed briefly in this chapter.

8.2 Cost effectiveness of mitigation

In a 2019 study, it was found that for every US$1 that communities spent on mitigation, there was a multiplier in savings for various infrastructure and for different natural hazards. Table 8-1 (NIBS, 2019) summarized findings for natural hazards and infrastructure considered in this report. Note that mitigation will result in significant savings; for US$1 spent on earthquake or flood mitigation for lifelines, savings of US$3-8 could be realized.

		Infrastructure	
	Case	**Buildings**	**Lifelines[15]**
Natural hazard	River flood	6:1	8:1
	Storm surge	7:1	No data
	Wind	6:1	7:1
	Earthquake	13:1	3:1

Table 8-1. Cost-benefit analysis for natural hazard mitigation (adapted from NIBS, 2019)

8.3 Local costs

Data from some infrastructure reconstruction for various infrastructure was collected from some of the subject countries; see Table 8-2[16].

Sector	Subsector	Typology	Country	Cost (XCD[17])	
Transport	Paved roads	Asphalt	Saint Vincent and the Grenadines	162,000	Per km of road
		Concrete		1,500,000-1,800,000	
	Bridges	Precast concrete girder		1,000-2,000	Per m^2 of deck
Buildings	School	Reinforced concrete		150-1,250	Per m^2 of floor
	Hospital	Reinforced concrete		1,300-1,800	
	Government	Reinforced concrete		2,800	
	Hospitality, hotel	Reinforced concrete		200-	

Table 8-2. Example of infrastructure reconstruction cost in the Caribbean (various)

[15] Power, water, and transport infrastructure
[16] Retrofitting cost data was not available
[17] Eastern Caribbean dollar

8.4 Life-cycle cost analysis

8.4.1 Overview

For infrastructure, the use of initial capital cost alone as a decision-making criterion on whether or not to undertake mitigation is not representative. To assess the cost and savings associated with improvements, it is important to perform engineering economic calculations based on a long horizon that is typically several decades and accounts for all future expenditures during the service life of infrastructure.

For infrastructure projects, Life-Cycle Cost Analysis (LCCA) is a better tool. LCCA is defined as performing analysis and comparing the merits of various alternatives. In the context of this report, there are two alternatives: i) do nothing; and ii) undertake mitigation. LCCA allows for a rational and structural approach in deciding which, if any, measures should be undertaken. The general approach is to examine all the costs and associated savings throughout the analysis time and account for the discount rate, defined as the net difference between interest and inflation rates. The option with the lowest net cost is then considered the optimal choice and can be assigned a higher priority than the options that have higher net costs associated with them.

8.4.2 Application to paved roads

In the U.S., most state transportation departments use LCCA for design of paved roads. FHWA (1998) provides the basic framework and procedure for conducting LCCA. The FHWA approach is primarily focused on future recurring costs due to wear and tear, because better constructed (or strengthened) roads are expected to perform better than existing roadways in natural hazards (primarily flooding). Thus, similar approaches could be utilized to assess the efficacy of resilient design. The FHWA LCC analysis is incorporated as part of the pavement design software. The FHWA guidelines recommend the following:

- Inclusion of both agency and user costs
- Minimum analysis period of 35 years
- Development of activity list (maintenance strategy) for roadways
- Agency costs, including initial engineering, contract administration, construction costs, future routine maintenance costs, resurfacing, rehabilitation, and traffic maintenance costs
- User costs are the differential costs to the travelers between the alternatives and include delay costs, crash costs, vehicle operating costs, and delay (time) costs.

Batouli et al. (2017) conducted a LCCA of pavement designs. Figure 8-1 presents the cumulative cost per unit length of road for the two alternatives. Note that the flexible pavement has a lower initial cost but considerably higher maintenance cost and two additional rehabilitation (re-surfacing) costs at 20 and 40 years in the design life. Note that the LCCA shows that the concrete pavement, although at a higher initial cost, is a more cost-effective solution than keeping the existing asphalt surface; in other words, mitigation pays for itself at most within the first 20 years. If there are earlier natural hazards, such as flooding, then this time is shrunk even more.

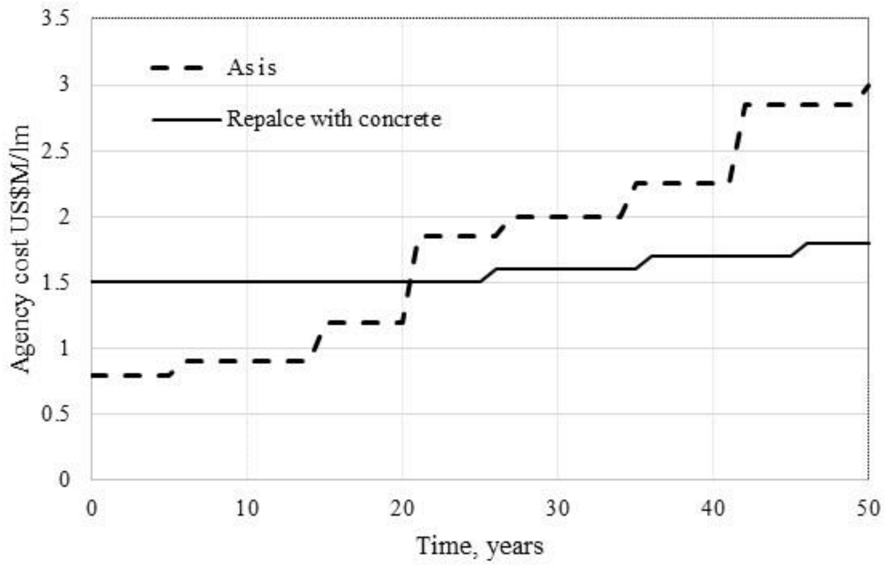


Figure 8-1. LCCA for example paved road (adapted from Batouli et al., 2017)

8.4.3 Application to bridges

AASHTO (2019) presents the framework for bridge LCCA. Agency costs are defined as construction, maintenance, rehabilitation, and replacement costs. User costs relate to the functionality of the bridge and include load posting, closures, higher accident rates, longer travel times, and higher vehicle operation costs to use detours. In addition, the risk vulnerability can be included in the cost model due to vulnerability to earthquakes and other natural disasters.

Hatami and Morcous (2013) performed LCCA for bridges in the U.S. State of Nebraska. A bridge was chosen to assess the efficacy of applying overlay to the bridge deck (thus the two analysis conditions were either “do nothing” or “mitigate”). The key properties of the bridge are summarized in Table 8-3. This bridge is similar to the types of structures found in the subject countries. As shown in Figure 8-2, the overlay adds an initial construction cost, but the option becomes cost-efficient approximately 18 years into the life of the bridge. For bridges, the typical service life is 50 to 100 years and thus, this mitigation is cost-effective. Note that user costs were not included in analysis. Inclusion of user costs would make mitigation even more cost-effective.

Item	Value
Length, m	78
Width, m	14
Spans	3
Location	Urban
Superstructure	Steel girder with concrete deck

Table 8-3. Key properties of the bridge (adapted from Hatami and Morcous, 2013)

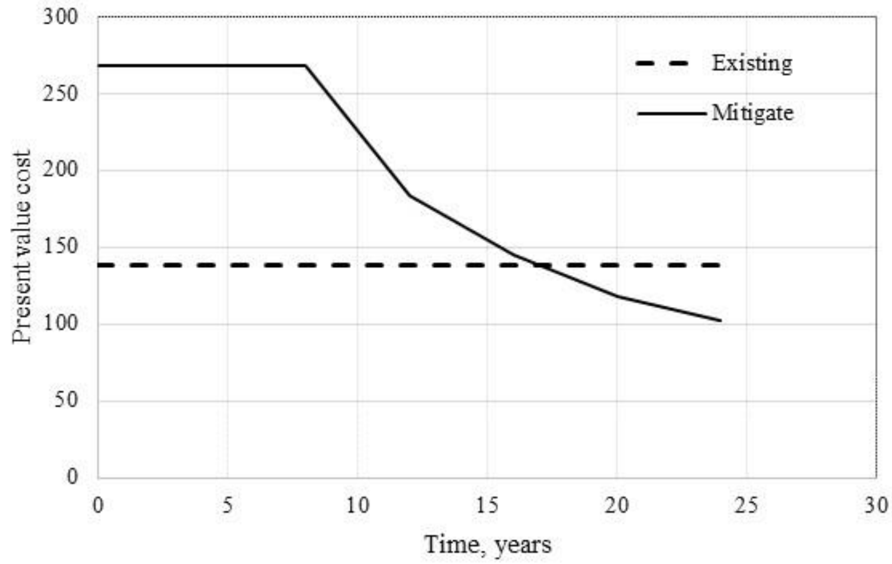


Figure 8-2. LCCA for example bridge (adapted from Hatami and Morcous, 2013)

8.4.4 Application to buildings

Porter et al. (2004) examined the average annual loss (AAL) for reinforced concrete buildings. These structures have similar construction to buildings in Caribbean. Figure 8-3 presents the mean seismic vulnerability functions. Note that for a PGA of 0.3-0.4 (note that this is similar to the level of seismicity in the Caribbean; refer to Section 2.2.1), the mean damage factor is reduced by a factor of approximately nearly ten when earthquake-resilient mitigation is implemented. The computed AAL for the buildings are 1.14%, and 0.15%, respectively. Thus, there is significant reduction in the expected annual losses once mitigation is undertaken.

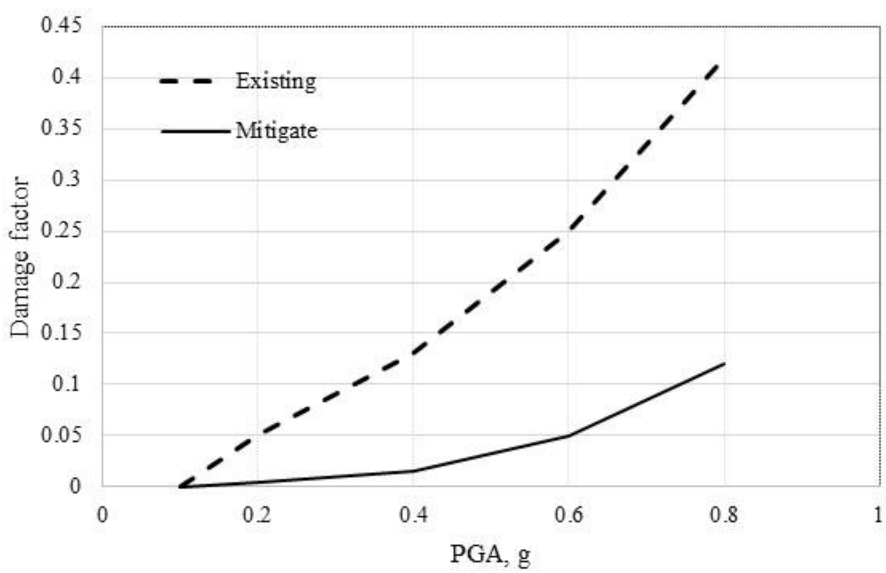


Figure 8-3. Mean seismic vulnerability function for buildings (adapted from Porter et al., 2004)

8.4.5 Application to energy sector

De Leon et al. (2009) examined the life-cycle cost functions for substations and transmission towers in Mexico subject to wind loading. They noted the following:

[I]t is observed that the scenario "with" the additional recording implies the most economical option because, as a result of the improvement on prediction, better design specifications are developed and the facilities reliabilities are enhanced. As the potential losses increase, the economical advantage also increases. Therefore, the critical facilities should have more closely and systematic recording and monitoring.

Loo et al. (2010) investigated the LCCA of switchgears. They noted over time, a lower percentage of failure (and lower LCCA) is obtained when preventive maintenance was included; see Figure 8-4.

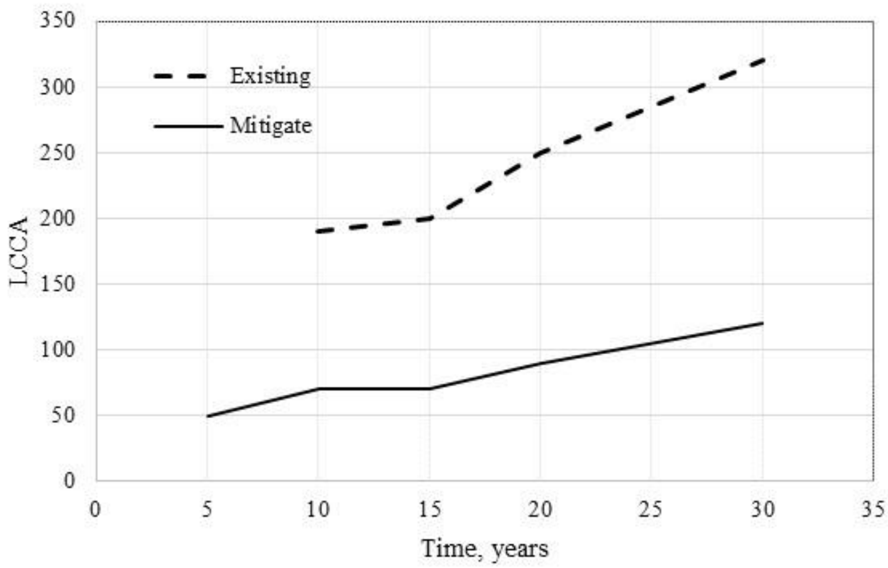


Figure 8-4. LCCA functions for energy sector (adapted from Loo et al., 2010)

8.5 Project contracting

8.5.1 Overview

The scope of retrofitting of infrastructure assets in the subject countries is expansive, will take a number of years, and will be resource-intensive. Examples of innovative contracting options that provide means for cost-sharing and (some) risk transfer are presented herein.

8.5.2 Design Bid Build (DBB)

In DBB, the work is performed sequentially and is the most common type of contracting. The agency or their consultants design the infrastructure. Plans and design criteria are then advertised in a bid package. The interested contractors will submit bids for the construction and the selection of a successful bid would be based on criteria developed by the agency.

Some disadvantages of DBB are (Caltrans 2020):

- Lack of input from construction industry could lead to more expensive design
- If the selection is based on lowest bid alone, the quality of construction might not be optimal, and a more extensive agency inspection program would be required.
- There are no incentives for contractors to expedite the project or provide higher quality product.

8.5.3 Design Build

In design-build, there is a single contract between the project owner and a design-build contractor who is responsible for both design and construction of an infrastructure retrofit project. The agency sets the project requirements, such as design parameters, expected timeline for construction milestones, and quality performance criteria. A comparison between project schedule for DBB and design-build is presented in Table 8-4 (USDOT, 2006).

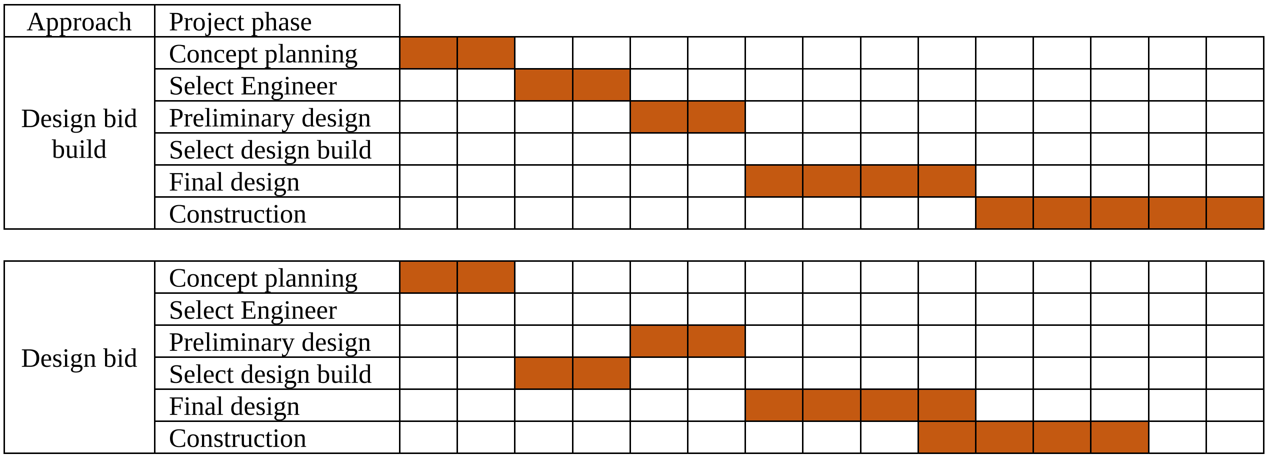


Table 8-4. Design build and DBB project approach (adapted from USDOT, 2006)

Table 8-5 presents the comparison between the two contracting approaches for key construction parameters (USDOT, 2006). Note that utilizing design-build will reduce construction time and cost without impacting the quality. The reduction in construction time is most pronounced and is most critical for infrastructure retrofit projects, as responsible agencies would prefer to minimize the service disruption time.

Parameter	Completion schedule	Project cost	Construction quality
Impact of design build vs. BDD	-14%	-3%	0%

Table 8-5. Example of a successful procurement process (adapted from USDOT, 2006)

8.5.4 Public-private partnership (P3)

In public-private partnership (P3), a private entity or developer takes part in financing the infrastructure project in return for monetary compensation based on contractual authorization to collect revenues. The private entity will be responsible for financing. The private entity will be responsible for financing, design, and construction.

FHWA encourages state transportation departments to enter into public-private partnerships as a means of securing additional financial resources. An example of a successful procurement process is presented in Table 8-6 (FHWA, 2019).

Stage	Component	Description
Pre-procurement	Feasibility Analysis	The P3 feasibility analysis helps determine whether a project can or should be implemented as a P3 project and, if so, the type of P3 to use and the related contractual and risk arrangements for project implementation.
	Advisors	The agency to engage its technical, legal, and financial advisors early in the procurement process.
	Market Soundings	The agency will often set up informal market sounding meetings or phone calls with interested firms to obtain information that firms may be reluctant to share in written responses
	National government support	Early engagement and coordination, prior to procurement, has been found to significantly help in ensuring efficient access to national programs and resources without impacting the desired procurement
	Local Support	Approval from or cooperative efforts by State or local agencies and other entities may be required for a P3 procurement.
	Industry Forum	A workshop prior to issuing the procurement documents helps to obtain comments and input from the private sector regarding the proposed procurement process.

	Confidentiality	Establish procedures ensuring confidentiality of information and documents
Initiation of Procurement	Selection process	Issuance of documents to the shortlisted proposers.
	Review process	Obtain input and comments from the shortlisted proposers, including one-on-one meetings with proposers
Pre-selection	Revisions	An opportunity for proposers to submit Alternative Technical Concepts
	Evaluation Committees	Evaluate technical and financial proposals on a pass/fail and qualitative basis.
Selection/	Selection process	A best value selection process or a qualifications-based selection process
	Engagement in discussions	Reservation of the right to engage in discussions with proposers and to request proposal revisions.
	Negotiations	Negotiate with the selected proposer prior to final award.
	Stipends	Pay stipends to all shortlisted firms if the procurement is cancelled.

Table 8-6. Example of a successful procurement process (adapted from FHWA, 2019)

8.5.5 Construction Manager/General Contractor (CMGC)

Caltrans (2020) writes:

> *The CMGC project delivery method consists of two phases-design and construction.*
>
> *When the owner considers the design to be complete, the construction manager then has an opportunity to bid on the project based on the completed design and schedule. If the owner, designer and independent cost estimator agree that the contractor has submitted a fair price, the owner issues a construction contract and the construction manager then becomes the general contractor.*
>
> *The contractor acts as the consultant during the design process and can offer constructability and pricing feedback on design options and can identify risks based on the contractor's established means and methods. As noted earlier, this process also allows the owner to be an active participant during the design process and make informed decisions on design options based on the contractor's expertise.*

The advantages of Construction Manager/General Contractor (CMGC) contracting include:

- Encourages innovations as contractor and owner work together to devise design and construction solutions.
- Improved design. Contractor reviews and provides constructability review.
- Risk mitigation. Obstacles not known during design in DBB projects will be brought up by the owner-contractor team.
- Expedited delivery. Project schedule is optimized as potential issues are discovered and resolved during the design process.

8.6 Discussion

Infrastructure planners face competing economic constraints. Experience has shown that retrofitting projects are extremely cost effective and for every US$1 spent on mitigation, savings of US$6 to 13 are realized. To implement strengthening techniques, it is critical to conduct a feasibility study. Utilizing a database of local costs would provide an estimate of initial capital costs. Given the long service life of infrastructure, performing LCCA would provide a realistic cost and savings associated with undertaking the retrofitting measures. Innovative contracting options can be used by infrastructure owners to reduce construction time and share project risks with contractors.

9. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

9.1 Summary

9.1.1 Overview

Buildings, water, transport, and power infrastructure are key assets for the Caribbean. The continuous operation of these critical structures allows for continuous economic growth and societal wellbeing. Damage to the infrastructure results in direct economic losses, operation interruption, and has cascading effects on other infrastructure.

Buildings, water, transport, and power infrastructure have been vulnerable to natural disasters, like hurricanes, flooding, and earthquakes, in the past. Given its geographical location, the Caribbean countries in this study are vulnerable to a wide range of natural disasters; see Table 9-1.

Country	Hurricane	Earthquake	Flood
Antigua and Barbuda	4	3	
Bahamas	4		
Belize	4		4
Barbados	3		
Dominica	3		
Guinea		3	4
Grenada	*No data*		
Guyana			4
Haiti	6	4	4
Jamaica	4	2	
Saint Kitts and Nevis	5	2	
Saint Lucia	*No data*		
Sint Maarten	*No data*		
Suriname			4
Trinidad and Tobago		3	
Saint Vincent and the Grenadines	4		

Table 9-1. Relative importance of hazards in Caribbean countries based on mortality index (ISDR, 2009)

The subject countries have been exposed to a large number of hurricanes in recent years with significant economic consequences. For example:

- In Dominica, Hurricane Maria (2017), with maximum wind speeds of 275 km/hr, damaged the roof of almost every house, caused severe damage to roads and public buildings, downed power lines, and caused US$1.4 billion in damage.
- In The Bahamas, Hurricane Dorian (2019), with sustained wind speeds of 300 km/hr, caused at least 40 fatalities, and damage of over US$7 billion; see Figure 9-1.

Before

After

Figure 9-1. Impact of Hurricane Dorian in the Bahamas (ESRI, 2019)

9.1.2 Critical infrastructure

Table 9-2 presents the key infrastructure, typologies, and vulnerable components considered in this report. The table also identifies the expected governing natural hazards for various components.

Sector	Subsector	Type	Components	Governing Hazard(s)
BUILDINGS	Schools, Hospitals, Government buildings, Emergency centers, Hospitality and hotels	URM	Wall	Earthquake
			Roof	Wind
		RC/CF with infill wall	Shape and figure	Earthquake
			Wall	Earthquake, Flood
			Roof	Wind
		RC frame	Shape and figure	Earthquake
			Ductility	Earthquake
			External wall	Wind
		Steel frame	Stiffness	Earthquake, Wind
			Strength	Earthquake
			Roof	Wind

Sector	Subsector	Type	Components	Governing Hazard(s)
		Steel frame high-rise	Stiffness	Earthquake, Wind
			Ductility	Earthquake
			Cladding	Wind
		Wood frame	Roof and foundation	Wind, Flood
			Elevation	Flood
		Foundation		Earthquake liquefaction
TRANSPORT	Roads	Asphalt paved	Subsurface	Flood
			Surface	Flood
	Bridges	Concrete girder	Columns	Earthquake
			Joints	Earthquake
		Steel truss	Slender members	Earthquake
			Riveted connections	Earthquake
		Steel girder	Bearings	Earthquake
			Cross frames	Earthquake
POWER	Power plants	RC low rise	Structural details	Earthquake
			Nonstructural components	Earthquake
			Grade construction	Flood
	Substations	Control buildings	Building vulnerability	Earthquake
			Grade construction	Flood
		Equipment	Unanchored equipment	Earthquake
			Lack of seismic components	Earthquake
	Transmission lines	Metal lattice main lines	Metal lattice poles	Wind
			Transmission lines	Wind
WATER	Water treatment plant and Wastewater treatment plant	Plant building (Masonry/RC frame)	Wall and ductility	Earthquake
			Roof and external wall	Wind
			Envelope and elevation	Flood
		Water storage	Foundation and anchorage	Earthquake
		Equipment (mechanical and electrical)	Foundation	Earthquake, Wind
			Elevation and flood proofing	Flood
		Foundation		Earthquake liquefaction
	Pipelines	Jointed concrete pipes	Joints	Earthquake. liquefaction
		Ductile iron pipes	Joints and welded flanges	Earthquake

Table 9-2. Matrix for the various infrastructure sectors

9.1.3 Vulnerable components

Table 9-3 presents the components that have experienced the most damage in past events and that are considered in this report.

Sector	Subsector	Type	Vulnerability	Hazard
BUILDINGS	Schools, Hospitals, Government buildings, Emergency centers, Hospitality and hotels	URM	Out-of-plane capacity	Earthquake
			In-plane capacity	Earthquake
			Roof connection	Wind
		RC/CF with infill wall	Structural irregularity	Earthquake
			Infill wall connection	Earthquake
			Roof connection	Wind
			Building envelope	Flood
		RC frame	Structural irregularity	Earthquake

Sector	Subsector	Type	Vulnerability	Hazard
			Non-ductile detailing	Earthquake
			External wall	Wind
		Steel frame	Stiffness	Earthquake, Wind
			Structural capacity	Earthquake
			Roof connection	Wind
		Steel frame high-rise	Stiffness	Earthquake, Wind
			Non-ductile detailing	Earthquake
			External curtain wall	Wind
		Wood frame	Roof connection	Wind
			Foundation ties	Wind, Flood
			Elevated construction	Flood
		All	Foundation	Earthquake liquefaction
TRANSPORT	Roads	Asphalt paved	Subsurface	Flood
			Surface	
	Bridges	Concrete girder	Columns	Earthquake ground motion
			Foundation	
			Joints	
		Steel truss	Slender members	
			Riveted connections	
		Steel girder	Bearings	
			Cross frames	
		Steel bridges	Welded connection details	Wind
		Suspension bridge	Bridge stability	Wind
		All	Foundation	Earthquake liquefaction
POWER	Substations	Transformers	Inadequate anchorage	Earthquake ground shaking
		Support frames	Low stiffness and damping	Surface
		Eclectic components	Inadequate design	Columns
	Transmission towers	Steel tower	Slender members	Wind and earthquake shaking
		Transmission lines	Vibration	Wind
		Foundation	Inadequate capacity	Earthquake liquefaction and flood
WATER	Water treatment plant and Wastewater treatment plant	Plant building (Masonry/RC frame)	Out-of-plane capacity	Earthquake
			In-plane capacity	Earthquake
			Non-ductile detailing	Earthquake
			Roof connection	Wind
			External wall	Wind
			Building envelope	Flood
			Elevated construction	Flood
		Water storage	Foundation	Earthquake
			Anchorage	Earthquake
		Equipment (mechanical and electrical)	Foundation tie	Earthquake, Wind
			Elevating equipment	Flood
			Dry floodproofing	Flood
		All	Foundation	Earthquake liquefaction
	Pipelines	Pipe	Brittle pipes	Earthquake ground shaking Earthquake liquefaction
		Joints	Joints with limited deformation capacity	Earthquake ground shaking

Sector	Subsector	Type	Vulnerability	Hazard
				Earthquake liquefaction

Table 9-3. Vulnerable components of the various infrastructure and the corresponding hazards

9.1.4 Design codes and standards

The Caribbean Uniform Building Code (CUBiC) was developed within and by the Caribbean community in 1985, and it includes structural design requirements for gravity, wind, and seismic load (CARICOM, 1985). Since the degree of enforcement of this building code varies by country, some countries of the Caribbean have adopted this code as part of the design process, but some treat this as a supplemental document.

In Trinidad and Tobago, a multi-year bridge assessment and reconstruction project was completed in the early 2010s (Khan-Kernahan, 2013). As part of this project, 96 bridges underwent rapid assessment. From that pool, 70 bridges were selected for detailed assessment, 40 of which were selected for reconstruction. These reconstructed bridges were designed by international engineering consultants and the bridge design followed the American Association of Highway and Transportation Officials (AASHTO) design procedures in effect at the time.

ASCE 113 (2007) provides design criteria for power infrastructure structures. Although intended for the U.S. and North America, the document is used worldwide, including in the Caribbean. ASCE 113 wind design is based on provisions of ASCE 7. Similarly, the Pan American Health Organization (PAHO) has developed wind speeds to be used with wind provisions of ASCE 7 (2019).

The buildings and equipment for water treatment plants (WTP) and wastewater treatment plants (WWTP) in the Caribbean are designed and constructed by utilizing international design codes or Caribbean building codes. The construction of water supply facilities refers to country codes in addition to the codes of the U.S. or Europe.

9.1.5 Strengthening and resilience measures

Table 9-4 presents examples of proposed strengthening measures for the vulnerable components identified for each infrastructure sector.

Sector	Subsector	Type	Resilience Measures
BUILDINGS	Schools, Hospitals, Government buildings, Emergency centers, Hospitality and hotels	URM	Install steel brace or strongback and anchors
			Add or overlay RC shear wall
			Improve connection anchors of roof and wall with new bond beam
		RC/CF with infill wall	Balance lateral-force resisting system by adding RC shear wall
			Improve connection anchors of infill wall and diaphragm
			Improve connection anchors of roof and wall
			Install flood shield on door or window
		RC frame	Balance lateral-force resisting system by adding RC shear wall
			Overlay FRP sheet, steel jacket or RC jacket Add seismic isolation
			Improve connection of external wall
		Steel frame	Add steel braced frame or shear wall frame for stiffness
			Improve strength by adding new braces/walls or moment frames.
			Improve connection elements of roof and frame

Sector	Subsector	Type	Resilience Measures
		Steel frame high-rise	Add steel braced frames or shear wall frame for stiffness
			Improve beam-column connection for ductile behavior Add energy dissipation damper
			Improve external wall attachment details
		Wood frame	Improve connection anchors of roof and wall
			Strengthen foundation anchor bolt and stiffener
			Construct floodwall or floodgate
		All	Enlarge spread footing Add pile foundation Install soil grouting or stone/gravel columns
TRANSPORT	Roads	Asphalt paved	Compact subsurface Provide drainage
			Use concrete surface Use thicker asphalt
	Bridges	Concrete girder	Add cable restrainers
			Add abutment seat extenders
		Steel truss	Stiffen members Add BRB
			Replace riveted connections with bolted connections
		Steel girder	Place cable restrainers at bearings
			Upgrade cross frames with BRB
		Steel bridges	Improve welded connection detailing
		Suspension bridge	Increase torsional stiffness
		Columns	Add steel casing Add FRP casing Add pierwalls
		Foundation	Add isolation casing Add new piles and enlarge foundation Mitigate for liquefaction and scour
POWER	Substations	Transformers	Improve anchorage Base isolation
		Support frames	Stiffen the frame Wire rope dampers Spring dampers
		Eclectic components	Use of qualified components Flexible connectors
	Transmission towers	Steel tower	Retrofit slender members Add damping
		Transmission lines	Add vibration damping
		Foundation	Add new piles
WATER	Water treatment plant and Wastewater treatment plant	Plant building (Masonry/RC frame)	Install steel brace or strongback and anchors
			Add or overlay RC shear wall
			Overlay FRP sheet, steel jacket or RC jacket
			Improve connection anchors of roof and wall
			Improve connection of external wall
			Install flood shield on door or window
			Construct floodwall or floodgate
		Water storage	Add new internal/external concrete foundation
			Strengthen anchorage between storage and foundation

Sector	Subsector	Type	Resilience Measures
		Equipment (mechanical and electrical)	Improve foundation anchorage and stiffener
			Elevate equipment on new frame Relocate equipment at upper location
			Install watertight barrier
		All	Enlarge spread footing Add pile foundation Install soil grouting
	Pipelines	Pipe	Replace brittle pipes with ductile pipes (DI, HDPE etc.)
		Joints	Add joints with large deformation capacity

Table 9-4. Vulnerable components of the various infrastructure and the corresponding hazards

9.1.6 Cost considerations

Figure 9-2 presents the probability of the infrastructure being in a given damage state when subjected to an expected natural hazard. Note that once retrofitting, as outlined above, is implemented, the probability of the infrastructure experiencing significant damage or collapse is reduced from approximately 80% to approximately 20%. Such savings should be considered when planning for and allocating the initial capital costs of strengthening. To obtain an estimate of retrofit and reconstruction costs, an extensive literature survey was conducted, and local sources were contacted. Table 9-5 presents a summary of the findings related to the most critical components of infrastructure in the subject countries.

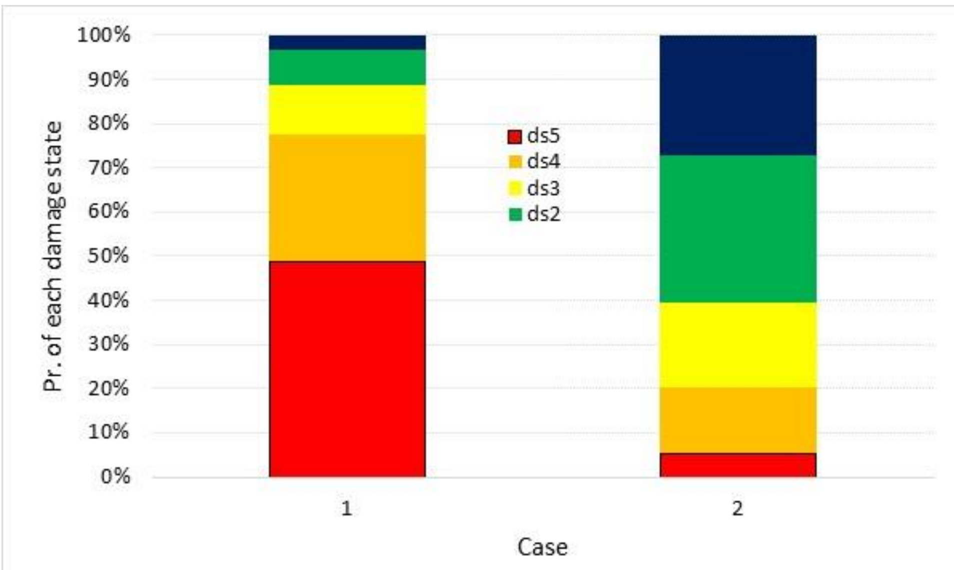


Figure 9-2. Distribution of damage states

Subsector	Hazard	Vulnerable Component	Resilience Measure	Cost, % of initial cost (Est.)	
				Improvement Cost	Vulnerability Reduction
Schools, Hospitals, Government buildings, Emergency centers, Hospitality and hotels	Earthquake shaking	Wall out of plane	Add wall bracing (anchor and strongback)	15%	40%
		In plane	Add RC shear wall or Steel brace	20%	60%
		Structural irregularity	Add Shear wall, Moment frame or Braced frame	15%	40%
		Ductility	Add Column jacketing or Beam retrofit	10%	30%
		Stiffness and capacity	Add Steel brace or Steel plate shear wall	15%	50%

Subsector	Hazard	Vulnerable Component	Resilience Measure	Cost, % of initial cost (Est.)	
				Improvement Cost	Vulnerability Reduction
		Energy absorption	Add Seismic isolation or Energy dissipation damper	40%	80%
	Earthquake liquefaction	Foundation	Enlarge Spread footing or Add pile foundation	40%	80%
		Soil	Install Soil grouting or Stone/gravel columns	30%	80%
	Wind	Roof connection	Improve Roof connection anchor	10%	50%
		External curtain wall and wall	Improve Wall connection attachment	10%	30%
		Foundation connection	Add Connection anchor bolts and stiffeners	10%	50%
	Flood	Building envelope	Install flood shield	10%	80%
		Elevated construction	Elevate floodwall	15%	80%
		Foundation tie	Strengthen anchoring to foundation	10%	50%
Bridges	Earthquake ground motion	Columns	Add FRP or steel casing	10%	60%
		Bearing	Add cable restrainers	2%	50%
		Abutment, pier seats	Add concrete bolsters	5%	50%
		Steel cross frames	Fatigue-resistant details	10%	80%
	Flood	Foundations, abutments	Add riprap	5%	20%
	Flood or liquefaction		Add new piles	40%	80%
Paved roads	Flood	Paved surface	Add sealant	2%	20%
			Strengthen (thicker pavement)	20%	50%
			Concrete surface	40%	80%
Substation components	Earthquake	Transformer	Improved anchorage	5%	20%
		Equipment	Qualified components	10%	40%
		Equipment	Spares	10%	--
		Support frames	Add passive dampers	5%	20%
Transmission system	Wind	Transmission tower	Strengthen steel members	20%	50%
		Transmission line	Add vibration dampers	5%	50%
Foundations	Flood or liquefaction		Add micro piles	40%	80%
WTP/WWTP : Plant building (Masonry/RC frame), Water storage, Equipment	Earthquake shaking	Wall out-of-plane (Building)	Add wall bracing (anchor and strongback)	15%	40%
		In-plane (Building)	Add RC shear wall	20%	60%
		Ductility (Building)	Add column jacketing	10%	30%
		Foundation (Water storage)	Add concrete foundation	40%	50%
		Anchorage (Water storage)	Strengthen anchorage	10%	30%

Subsector	Hazard	Vulnerable Component	Resilience Measure	Cost, % of initial cost (Est.)	
				Improvement Cost	Vulnerability Reduction
		Foundation tie (Equipment)	Improve foundation tie	10%	20%
	Earthquake liquefaction	Foundation (All)	Enlarge spread footing or add pile foundation	40%	80%
		Soil (All)	Install soil grouting	30%	80%
	Wind	Roof connection (Building)	Strengthen roof connection	10%	50%
		External wall (Building)	Improve wall connection	10%	30%
		Foundation tie (Equipment)	Improve foundation tie	10%	20%
	Flood	Building envelope (Building)	Install flood shield	10%	80%
		Elevated construction (Building)	Construct floodwall	15%	80%
		Elevating (Equipment)	Elevate equipment	20%	80%
		Dry floodproofing (Equipment)	Install watertight barrier	15%	80%
Underground pipelines	Earthquake Shaking and liquefaction	Pipes	Replace with ductile pipes Trenchless technologies	40%	80%

Table 9-5. Estimated monetary savings for implementing infrastructure resilient measures in the Caribbean

9.2 Conclusions

Based on the review of infrastructure typology and geographical distribution, hazard maps, recorded past damage, the construction practice, and relative costs, the following is noted:

- The 16 Caribbean countries in focus in this report are geographically located in areas that can experience a number of natural hazards, which could damage buildings, water, transport, and power infrastructure. The consequences include not only direct structural damage, but also secondary economic losses due to loss of revenue from inoperability, delayed recovery, and cascading costs to other infrastructure.
- Efficacious and economically-feasible strengthening options are available. These measures rely on well-proven techniques and are suitable to the Caribbean countries, given the availability of material and skilled labor resources. Once implemented, such measures would reduce infrastructure vulnerability by multiples of the initial capital cost. New technologies can be utilized to implement robust solutions and enhanced resiliency.
- The service life of infrastructure is comprised of planning, design, construction, maintenance, and post-event monitoring. Although structural strengthening is a key component of infrastructure improvements, other phases should also be considered for a successful enhanced resiliency program. Devising and implementing a construction quality management plan would ensure that retrofits are built as designed and would add to the service life of infrastructure, reducing the need for corrective maintenance. Preventive maintenance and inspection would reduce the cost of expensive future repairs. Early-warning and post-event inspection can result in reduced delays and expedite the return to functionality.

- Creative financing measures and life-cycle cost analyses (LCCA) can be used to identify and mitigate project risks, accelerate project delivery, and coordinate the design and construction phases by incorporating input from experts in both fields.

9.3 Recommendations

Based on the review of hazard, infrastructure typology and vulnerability, and steps taken during data collection, the following is recommended

- Perform a review of local design and construction codes. Review international codes and prepare recommendations for local codes and standards. In particular, the design wind speed and flood levels might need modifications based on recent hurricanes and the detrimental effects of climate change.
- Local infrastructure data is sparse. In the appendices in this report, examples of data collection templates are provided. It is suggested that utility companies and other stakeholders conduct an inventory search of all critical assets and maintain such data in a searchable database. As part of this effort, screening of components can also be conducted to identify the most vulnerable assets for a prioritized retrofitting.
- Develop a retrofit guide and incorporate typical details (plans and specifications) for the most common retrofits.
- Implement a feasibility study for a selected sector and subsector. Preform probabilistic LCCA to identify the most effective retrofits. For the probabilistic LCCA, incorporate realistic initial capital costs and recurring costs.
- Region-wide organizations such as CDEMA can develop a post-event damage assessment program, prepare paper and electronic forms, train engineers to perform assessments, and maintain an updated database of trained professionals that can perform these tasks in the event of natural hazards.
- Program and undertake retrofitting for one of the countries in a selected sector. This program can serve as a pilot program for use in other countries with similar regional characteristics.

9.4 Future activities

As discussed in the report, the objective of this project was to perform an initial review of buildings, water, transport, and power sectors in the selected countries. The scope was further limited to structural design only, to existing construction, and to limited infrastructure in each subsector. As part of a follow-up project, it is recommended that the World Bank Group consider the following components:

- Perform a more in-depth analysis. Such activity would require more extensive data collection regarding infrastructure exposure and cost components.
- Using the collected data from local stakeholders as discussed above, develop exposure models and GIS maps for both hazard and exposure. Conduct multi-hazard risk analyses for infrastructure networks and obtain individual and aggregate losses (structures, repair cost, revenue loss, recovery time).
- Expand the subsectors for various infrastructure sectors. For example, ports and airports could be considered in future studies.
- Broaden the scope beyond structural design to include more detailed studies of both construction quality and maintenance programs, and costs associated with them.
- Extend the cost computations to include secondary and user cost, and recovery time.
- Because hurricanes and coastal floods are a major natural hazard for most countries, undertake an evaluation of flood protection (levees, embankments, etc.) and flood prevention (drainage, collection, etc.) structures in place for coastal areas. Consider improvements and resilience measures for these structures. Such measures provide an additional layer of protection against natural hazards for the infrastructure sectors considered in this report.

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APPENDIX A. REVIEW OF DATA

A.1 Date Collection template

As part of this project, a data collection template was developed. This template (see Table A.1) can be used to develop the exposure model for various infrastructure.

A.2 Collected data

As part of the activities, the local members of the project team used the abovementioned templates and collected site-specific data. Examples for the building sector are presented in Table A.2. As discussed in the main body of this report, a number of factors contributed to the difficulty of collecting country-specific data.

A.3 World Bank-supplied data

A Geojson file was supplied by the WBG. An assessment of the data is presented in Table A.3. Note that the WBG data provides invaluable information related to the geographical location and distribution of several assets in a number of the countries. However, data listed in Table A.1 needs to be collected in order to augment the geographical data to perform quantitative risk assessments for infrastructure sectors.

Power Sector: Powerplants

No	Country	Sector	Sub-sector	Date	Collected by	Lat., deg	Long., deg	Footprint, m^2	Capacity, kV	People served	Year built	Design code	Plans available	Material and framing	Construction cost	Annual maintenance cost
1		Power	Power plant													
2		Power	Power plant													
..		Power	Power plant													

Power Sector: Substations

No	Country	Sector	Sub-sector	Date	Collected by	Lat., deg	Long., deg	Footprint, m^2	Capacity, kV	People served	Year built	Design code	Plans available	Resilient equipment	Construction cost	Annual maintenance cost
1		Power	Substation													
2		Power	Substation													
..		Power	Substation													

Power Sector: Distribution Lines

No	Country	Sector	Sub-sector	Date	Collected by	Beg Lat., deg	Beg Long., deg	End Lat., deg	End Long., deg	Length, km	Capacity, kV	People served	Year built	Design code	Plans available	Pole type	Cost/km
1		Power	T&D														
2		Power	T&D														
..		Power	T&D														

Water Sector: Water Treatment Plants

No	Country	Sector	Sub-sector	Date	Collected by	Lat., deg	Long., deg	Footprint, m^2	Capacity	People served	Year built	Design code	Plans available	Primary material and framing	Construction cost	Annual maintenance cost
1		Water	WTP													
2		Water	WTP													
..		Water	WTP													

Water Sector: Wastewater Treatment Plants

No	Country	Sector	Sub-sector	Date	Collected by	Lat., deg	Long., deg	Footprint, m^2	Capacity	people served	Year built	Design code	Plans available	Primary material and framing	Construction cost	Annual maintenance cost
1		Water	WWTP													
2		Water	WWTP													
..		Water	WWTP													

Water Sector: Pipelines (underground)

No	Country	Sector	Sub-sector	Date	Collected by	Beg Lat., deg	Beg Long., deg	End Lat., deg	End Long., deg	Length, km	Capacity, m^3/sec	People served	Year built	Design code	Plans available	Pipe type and material	Cost/km
1		Water	UGP														
2		Power	UGP														
..		Power	UGP														

Transport Sector: Bridges

No	Country	Sector	Sub-sector	Date	Collected by	Lat., deg	Long., deg	Length, m	Material	Superstr.	Span, m	Skew, deg	Piers	Columns/pier	Vert clr, m	ADT	Year built	Design code	Cost/ m^2 deck	Inspection cycle
1		Transport	Bridge																	
2		Transport	Bridge																	
..		Transport	Bridge																	

Transport Sector: Primary Paved Roads

No	Country	Sector	Sub-sector	Date	Collected by	Beg Lat., deg	Beg Long., deg	End Lat., deg	End Long., deg	Length, km	Surface (AC concrete)	Avg. daily traffic	Year built	Design code	Cost/km road
1		Transport	Primary paved road												
2		Transport	Primary paved road												

..		Transport	Primary paved road												

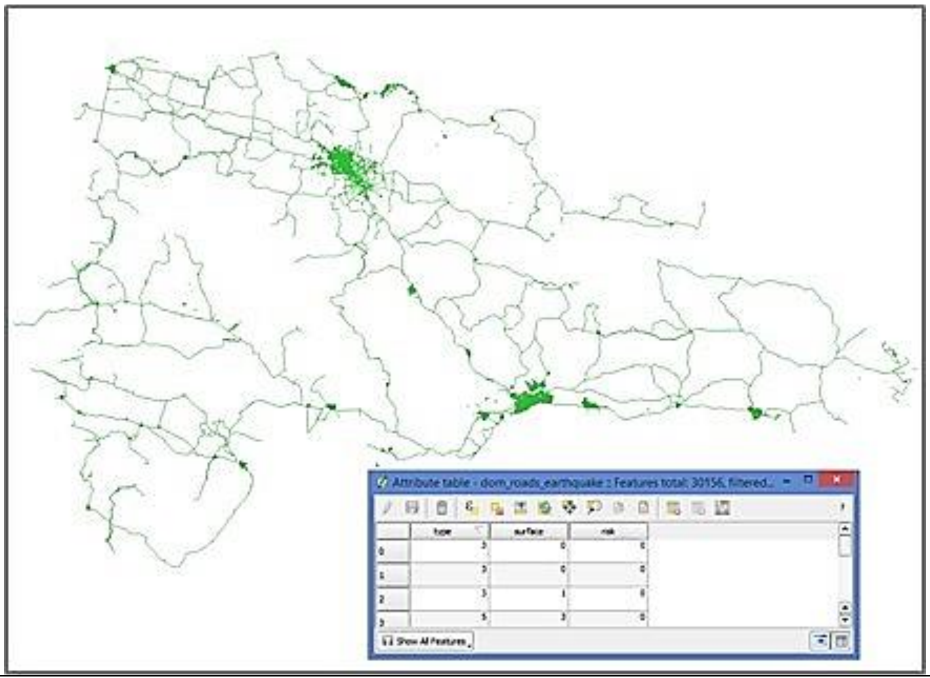
Buildings: Critical Facilities														
No	**Country**	**Sector**	**Type**	**Date**	**Collected by**	**Lat., deg**	**Long., deg**	**Footprint, m2**	**Stories**	**Framing type**	**Primary material**	**Occupants**	**Year built**	**Design code**	**Construction/Replacement Cost**
1		Buildings	School												
2		Buildings	School												
..		Buildings	Hospital												

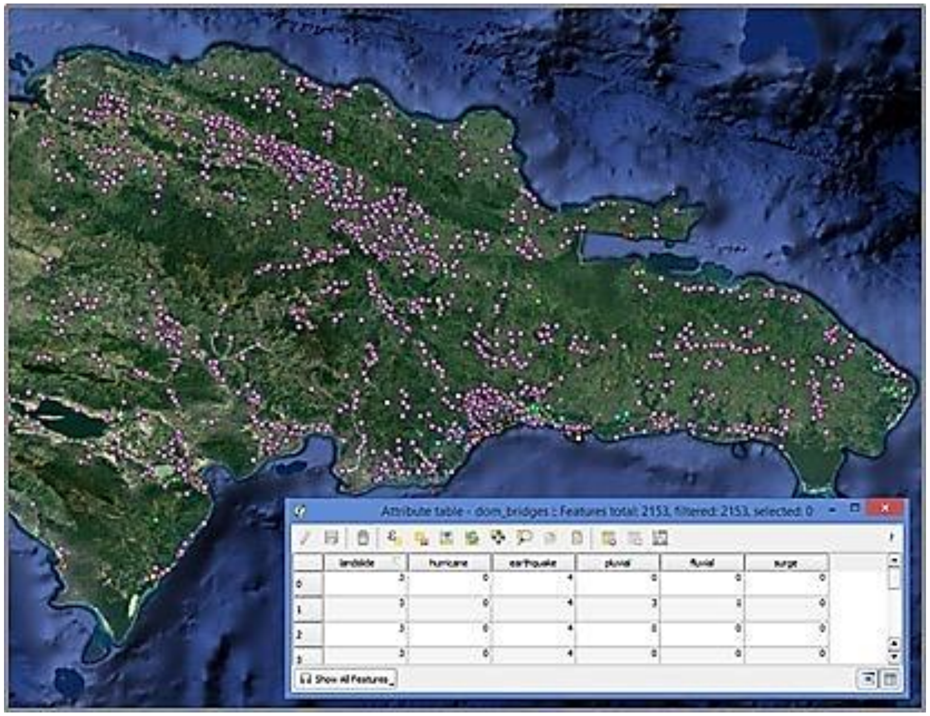
Table A.1. Date collection template

File name	For which component	Country	Category
Resilience in the Caribbean_ Natural Hazards Exposure Assessment and Areas for Future Work.pdf	Per Table 2A, Hazards for each country. Probably, can classify the countries per primary hazards. Per Table 2B, the target infrastructure information might be available in terms of "structural type".	All 16 countries	Hazard
13_TT_National_Building_Code_Overview-Thompson.pdf	An overview of TT codes (e.g., small buildings, Electricity, Driveway gates). Currently, there is no formal building codes in TT.	Trinidad & Tobago	Codes
Antigua.pdf	It looks a code exists in Antigua.	Antigua	Codes
Bahamas IMPROVING BUILDING STANDARDS CDB MEMBER COUNTRIES 2018 BAH 1.pdf	It looks a code exists in Bahamas (1st in 1971, 3rd in 2003).	Bahamas	Codes
Belize.pdf	It looks a code exists in Belize (1st in 2003, latest in 2017).	Belize	Codes
guide_to_dominicas_housing_standards-7.pdf LAWPhysical Planning Draft Building Code_Dominica.pdf	It looks this standard is mainly for Hurricane. This is based on 2002 Dominica Building Code. Also, 1996 Building Code (draft) in 1996.	Dominica	Codes
GUYANA_Building Code Presentation.pdf	It looks a code exists in Guyana.	Guyana	Codes
Jamaica Building Act 2018 NdaC pp Presentation.pdf	It looks Jamaican Building Code was consisted of application documents created in 2003 based on IBC.	Jamaica	Codes
bcqs-construction-market-report-2019.pdf	Page 10 shows the difference of construction cost for buildings per some Caribbean countries.	Several	Cost
Building Procedures in Belize.docx	Belize's construction cost (house?) is shown.	Belize	Cost
matthew_bahamas_.pdf	Hurricane damage on Essential buildings, Roads, Bridges, Water, Power, etc. of Bahamas at 2016 Hurricane Matthew.	Bahamas	Damage
Commonwealth of Dominica - Rapid Damage and Needs Assessment Final Report .pdf	Tropical Storm damage on Essential buildings, Roads, Bridges, Water, Power, etc. of Dominica at 2015 Tropical Storm Erika.	Dominica	Damage
H.Tomas-DANA (APESL).pdf	Hurricane damage on Public buildings, Roads and Bridges of St. Lucia at 2010 Hurricane Tomas.	St. Lucia	Damage
St Lucia Electricity Services Study.docx	Vulnerability study for Power infrastructure in St. Lucia.	St. Lucia	Vulnerability
Disaster Risk Reduction Country Doc Suriname.pdf	Major hazard for Suriname (looks flood).	Suriname	Hazard
CHARIM_Belize_NFHM_Methodology_FINAL.pdf COASTAL_FLOOD_HAZARD_BELIZE12016.pdf FLOOD_HAZARD_BELIZE2016.pdf	Flood Hazard Maps for Belize (CHARIM 2016)	Belize	Hazard
Dominica_Flash_Flood_Hazard_map_1.pdf Dominica_Landslide_susceptibility_Map.pdf	Flood Hazard Map and Landslide Susceptibility Map for Dominica (CHARIM)	Dominica	Hazard
SLUFloodReport.pdf SLULandslideReport.pdf St Lucia Landslide Susceptibility Map.pdf St_Lucia_Flash_Flood_Hazard_Map_1.pdf	Flood Hazard Map and Landslide Susceptibility Map for St. Lucia (CHARIM)	St. Lucia	Hazard

flood_risk_multi_trinidad.pdf flood_susceptibility_trinidad.pdf landslide susceptibility trinidad.pdf landslide_risk_multi_trinidad.pdf	Flood Risk Map, Flood Susceptibility Map, Landslide Susceptibility Map and Landslide Risk Map for Trinidad and Tobago (ODPM)	Trinidad & Tobago	Hazard
PGA(g)_RP=475years.jpg SeismicHazMaps_Summary_2011_06 .pdf	Seismic Hazard Map for Eastern Caribbean Countries (2011) - the north the Leeward Islands (from Anguilla to Dominica) and in the south the Windward Islands (from Martinique to Grenada), Barbados and Trinidad and Tobago	Eastern Caribbean Countries	Hazard

Table A.2. Samples of collected data for building sector

File name	Contents	Usefulness
Final Report_12June2020_v2.pdf	Resilience in the Caribbean: Natural Hazards Exposure Assessment and Areas for Future Work (12 June 2020)	- Assuming, at least, the hazard maps (for all hazards in GIS format) and the exposure model (might not contain much information) must exist. So, it might be useable for our project according to our SOW. If not, we might not want to ask, I guess.
Geojson files in "infrastructures-roads"	(1) This data includes only the risk result of the road model in the format of line (vector data, length, location, type, surface). (2) "risk" in the attribute table seems to be a hazard risk score (0 through 5) mentioned in Table 3 of the report. But, this is not 100% clear and it's needs to be confirmed. (3) "type" and "surface" are specified in the attribute table. "type" seems to be a road classification (primary, secondary, tertiary, earth, others) mentioned in report Page 8/60 but not 100% sure and needs to be confirmed. "surface" seems to be a cover of road (cement, asphalt, gravel, dirt) mentioned in report Page 41/60 but not 100% sure and needs to be confirmed. 	- If needed, have to confirm whether or not any other data (altitude, etc.) of "roads" model exist. - If needed, have to ask Hazard maps (for each hazards, like Map B of Figure 1 in the report), Damage map (for each hazard, like Map C of Figure 1 in the report), Probability map (for each hazard, like Map D of Figure 1 in the report), and fragility functions used for each hazard damage probability.

Geojson files in "infrastructures"	(1) This data includes the several risk scores (landslide, hurricane, earthquake, pluvial flood, fluvial flood, storm surge) of other infrastructure models (airport, bridge, hospital, port, power plant, and wastewater) in the format of point (vector data, location and type are available). (2) The scores in the attribute table according to hazard type seems to be a hazard risk score (0 through 5) mentioned in Table 3 of the report. But this is not 100% clear and it's needs to be confirmed. (3) "type" seems to be a classification for each asset (size, fuel, etc.) mentioned in report Appendix B (but not more specific data like height, structural type, footprint, etc.) but not 100% sure and needs to be confirmed 	If needed, have to confirm whether or not any other data (altitude, volume, footprint, structural type, etc.) of all infrastructure asset model exist.

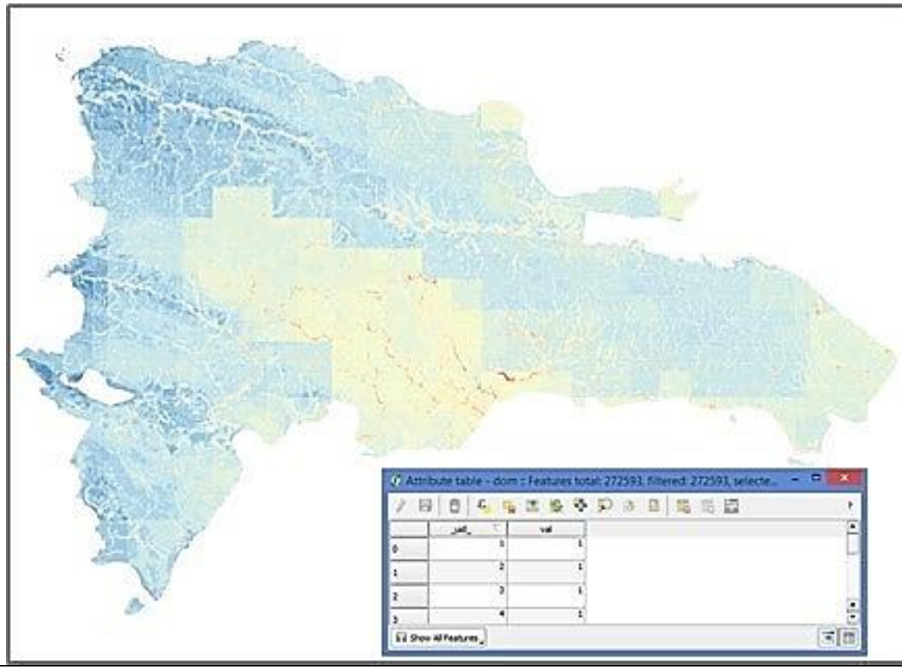
Geojson files in "multi-hazards"	(1) This data includes only the multi-hazard risk (all hazard risks were combined) and the risk score is between 0 and 30. (2) For example, Figure 3 in the report shows this type of multi-hazard risk map for Dominican Republic (the below map is also same one). (3) Other than that, no information is included.  	This seems to be combined with all hazard risks by a summation of each risk score with a weight factor (according to GPS coordinates), and it seems to be a qualitative risk result.

Table A.3. Review of WBG-submitted document

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