Hurricane Maria: A Harbinger of Imperative Changes to Caribbean Structural Design and Construction

Target: All Caribbean building and civil infrastructure operating at casualty, damage, and downtime probabilities equal to their acceptable loss probabilities

UNENGINEERED BUILDINGS (HOUSES)

HURRICANE MARIA DAMAGE ASSESSMENT

The following depicts photos taken in Dominica 3 days after the event and as part of a rapid visual assessment exercise. These are followed by sketches of observed failure modes, or hypothesized modes not directly observed, but expected to have occurred at other locations in Dominica.

It is important to note that addressing these specific failures does not necessarily mean failure may not occur elsewhere since zero failure is only possible if the demand-to-capacity (D/C) ratio for each element along the load path is less than unity. What is observed in the field is only for the element with the highest D/C so all others must be evaluated.

IMAGE 1 – AERIAL VIEW 3 DAYS AFTER SHOWING LAND STILL DRAINING

IMAGE 2 – DAMAGE TO FORESTRY AND COASTLINE

IMAGE 3 – RANDOM SAMPLE #1

IMAGE 4 – RANDOM SAMPLE #2

IMAGE 5 – RANDOM SAMPLE #3

IMAGE 6 – HOUSING DAMAGE OUTSIDE STADIUM #1

IMAGE 7 – HOUSING DAMAGE OUTSIDE STADIUM #2

IMAGE 8 – HOUSING DAMAGE OUTSIDE STADIUM #3

IMAGE 9 – HOUSING DAMAGE ON WATERFRONT TO SCOTT’S HEAD #1

IMAGE 10 – HOUSING DAMAGE ON WATERFRONT TO SCOTT'S HEAD #2

IMAGE 11 – HOUSING DAMAGE ON WATERFRONT TO SCOTT'S HEAD #3

IMAGE 12 – HOUSING DAMAGE ON WATERFRONT TO SCOTT’S HEAD #4

IMAGE 13 – HOUSING DAMAGE ON WATERFRONT TO SCOTT’S HEAD #5

IMAGE 14 – HOUSING DAMAGE ON WATERFRONT TO SCOTT’S HEAD #6

IMAGE 15 – TYPICAL ROOF DAMAGE – FASTENER PULLOUT AND SHEETING TEAROUT

IMAGE 16 –EXAMPLE OF NAIL PULLOUT

IMAGE 17 –EXAMPLE OF SHEETING TEAROUT

IMAGE 18 –EXAMPLE OF TIMBER OUT-OF-PLANE WALL FAILURE

DOMINICA HOUSING FAILURE MODES/CAUSES

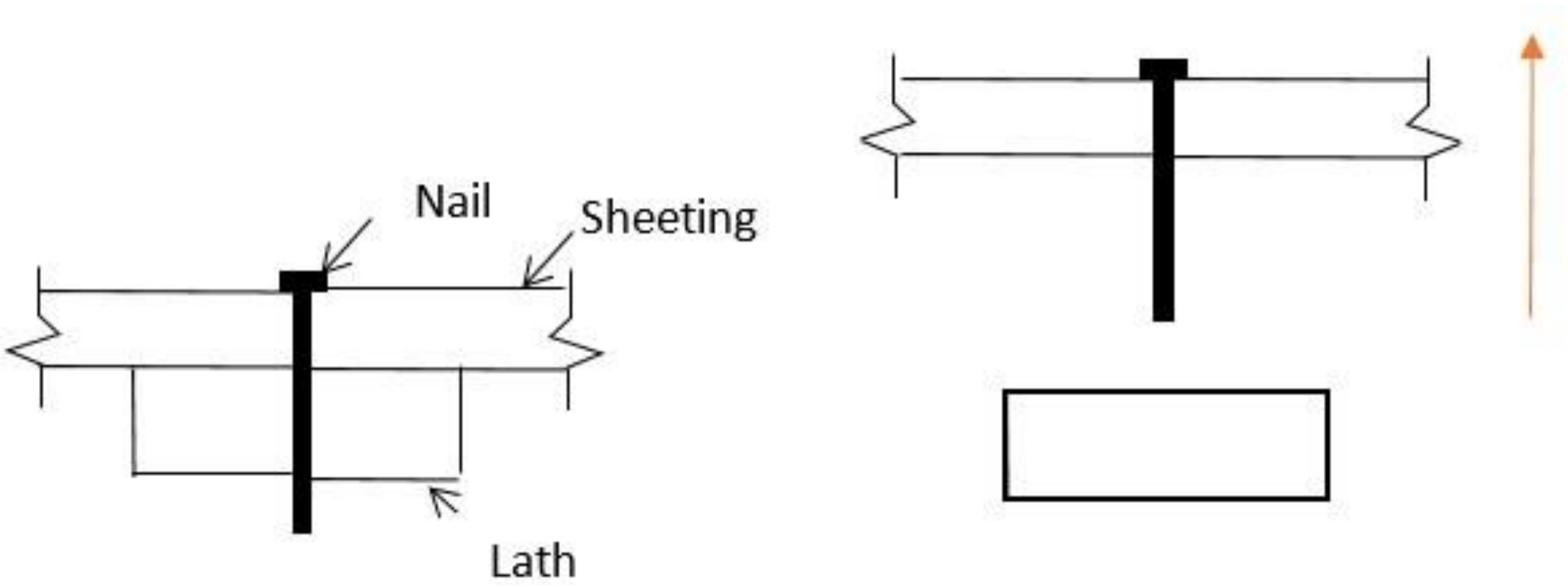
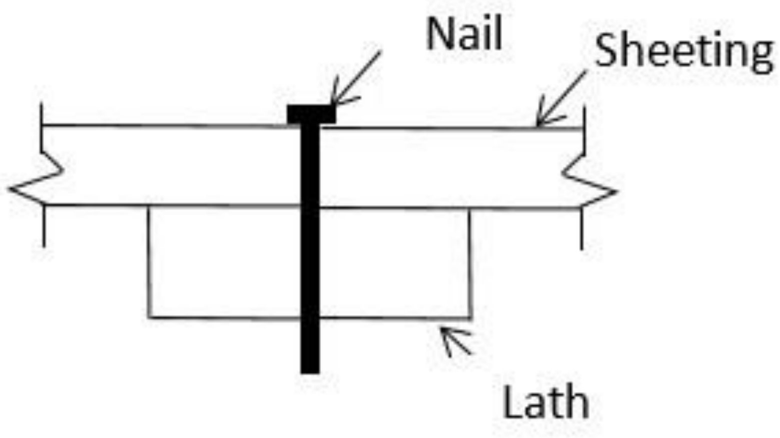


IMAGE 19 – Sheeting-to-Lath Connection
Failure Mode: Nail Pullout
Cause: Insufficient embedment; proper fastener not used; insufficient number of fasteners or inadequate installation



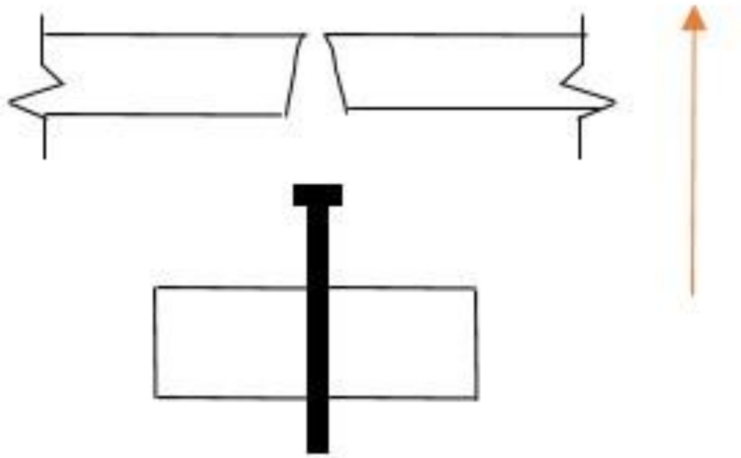


IMAGE 20 – Sheeting-to-Lath Connection

Failure Mode: Sheeting Tear-out

Cause: Sheeting too thin; washers non-existent or too small or too rigid; insufficient number of fasteners or inadequate installation

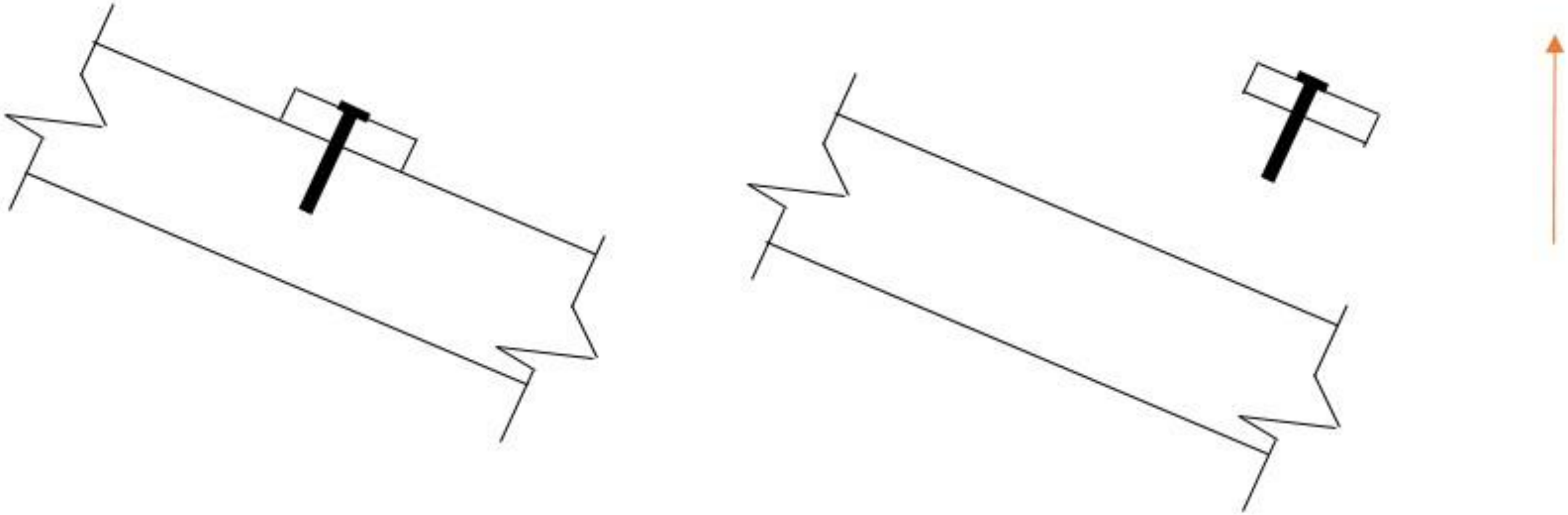


IMAGE 21 – Lath-to-Rafter Connection

Failure Mode: Nail Pullout

Cause: Insufficient embedment or proper strap not used or improperly used.

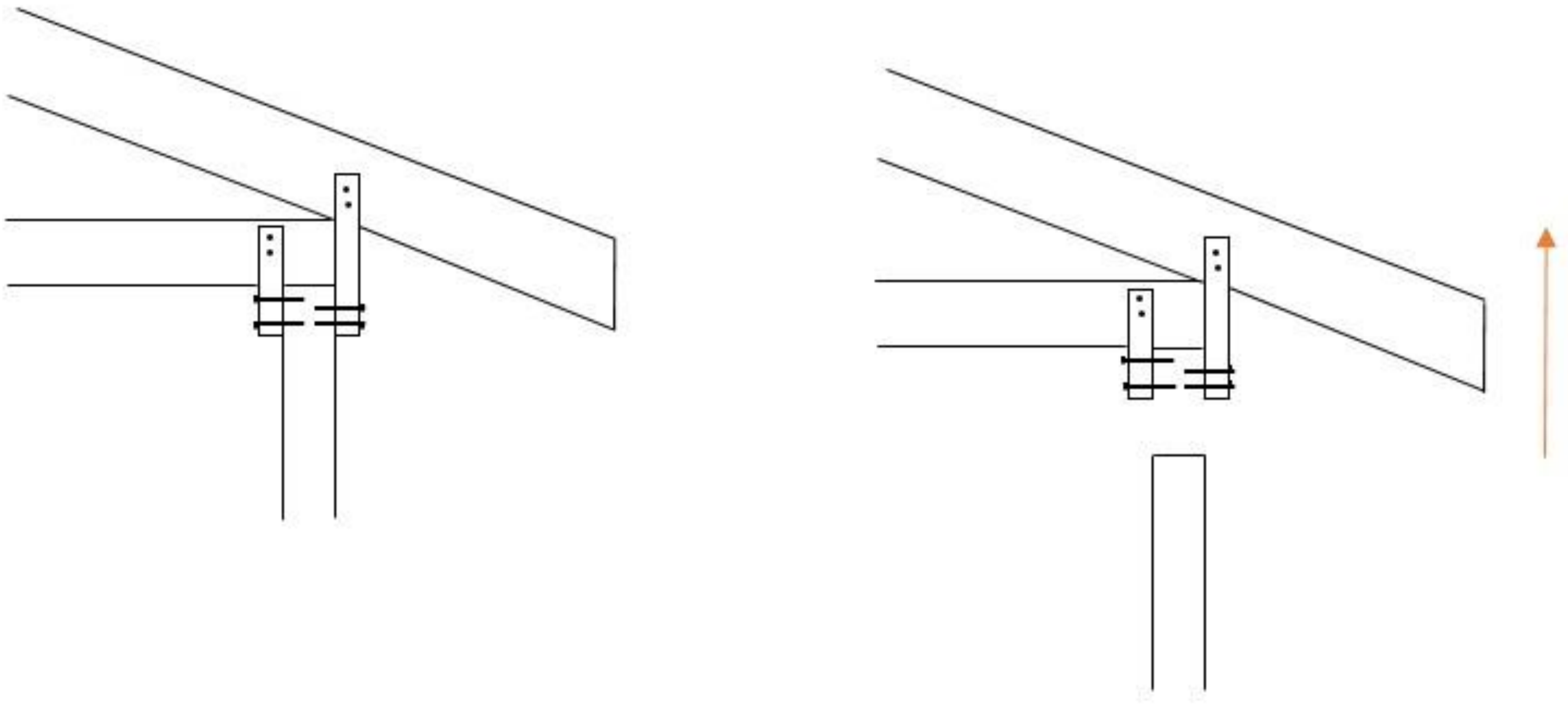


IMAGE 22 – Rafter or Truss-to-Wall Connection
Failure Mode: Assembly Pull-out
Cause: Insufficient tie-down capacity or proper strap not used or improperly used.

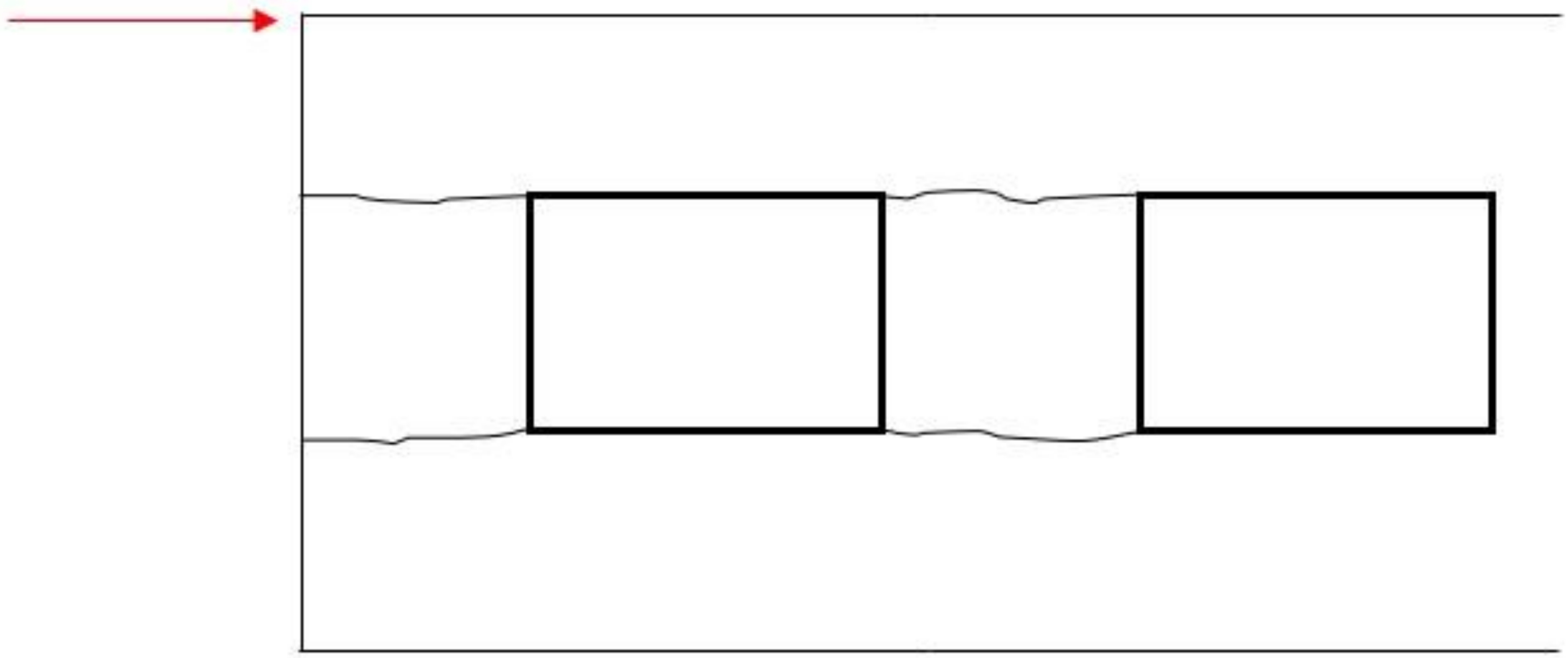


IMAGE 23 – Masonry Wall Piers
Failure Mode: In-plane bending (horizontal top and bottom cracking)
Cause: Insufficient block, joint mortar, or grout compressive strength or, too few rebars or rebar slippage due to inadequate compaction.

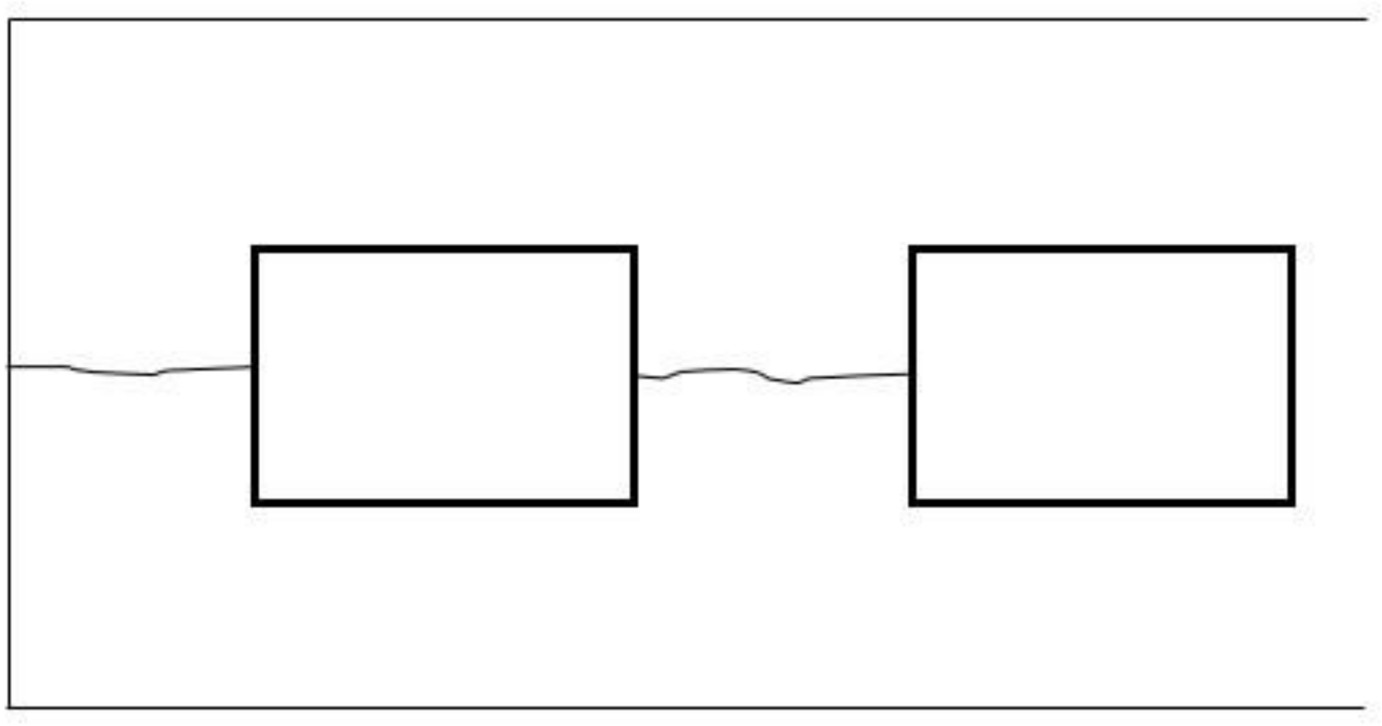


IMAGE 24 – Masonry Wall Piers

Failure Mode: Out-of-plane bending (horizontal mid-span cracking)

Cause: Insufficient block, joint mortar, or grout compressive strength or, too few rebars or rebar slippage due to inadequate compaction.

Other Failure Modes and Causes:
The following failure modes are also expected:

- Timber walls:
  i. Mode: Sliding
  ii. Cause: insufficient or improper fasteners to foundation stem walls
- Timber walls:
  i. Mode: Overturning
  ii. Cause: Ditto or insufficient weight
- Glazing:
  i. Mode: Fracture or collapse
  ii. Cause: Inadequate or non-existent shutters or, inadequate type or thickness of glass or framing
- Door connections:
  i. Mode: Tear-out
  ii. Cause: inadequate hinges or fasteners or framing or non-existent or improper shutters

The aforesaid failure modes and their causes due to hurricane Maria are well-known so there are no new technical lessons to be learned. In many cases where code prescriptions were applied adequate performance was achieved. However, it was also observed that there was substantial failure in many cases where construction appeared adequate. The wind speed was measured to be higher that the recommended design wind speeds at the time, so this may be a source of the observation, which would also exacerbate the extent of failure of those houses not consistent with proper construction.

Hence in terms of solutions for Dominica, an important need is the re-defining of the design wind speed. Furthermore, the following are required to reduce the risk of hurricane damage to housing to acceptable levels for new construction, and the retrofit of existing construction:

- For new construction: the implementation of the building codes
- For retrofitting existing houses: codes for new buildings can also be used as the basis for the retrofit design of the existing houses that were damaged, and those not damaged by Maria, but deemed to be at risk regarding future hurricanes

EVALUATION OF CARIBBEAN UNENGINEERED BUILDINGS (HOUSES)

The evaluation of Dominica houses under hurricanes was discussed in the previous section for both new construction and existing houses, as well as what solutions can be employed to reduce the risk to acceptable levels. The design and construction of houses in other Caribbean territories are similar to those of Dominica so the conclusions as regards hurricane-related deficiencies and how to address them are the same.

Events such as Hurricane Maria give the opportunity to examine in what other areas are the structures vulnerable, especially as regards natural hazards. As measures for reducing hurricane damage risk are being actively explored at this time, it is reasonable to attempt to also reduce risk due to the other prevalent Caribbean natural hazard – earthquakes. This is because: (1) the forces on the structure due to hurricanes and earthquakes are lateral forces hence the same type of lateral load resisting systems can be employed to simultaneously do both jobs, and (2) the negative consequences of earthquakes are much higher compared with hurricanes.

CARIBBEAN HOUSING EVALUATION RESULTS - SINGLE-STOREY HOUSES: FAILURE MODES/CAUSES

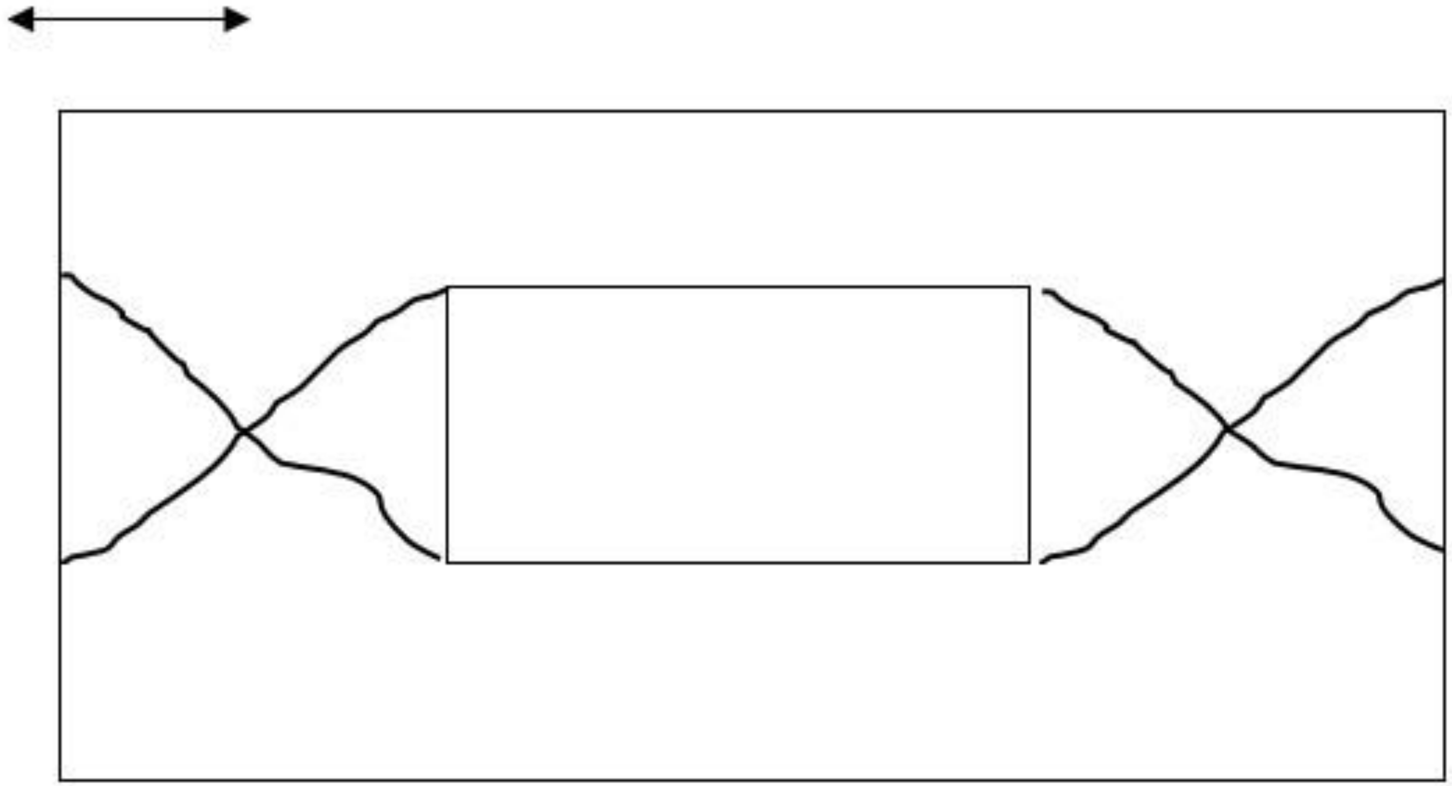


IMAGE 25 – 150 or 200 mm Masonry Wall Piers
Failure Mode: In-plane bending diagonal cracking
Cause: Inadequate horizontal reinforcement; poor unit, joint mortar, and grout compressive strength; possibly excessive vertical rebar.

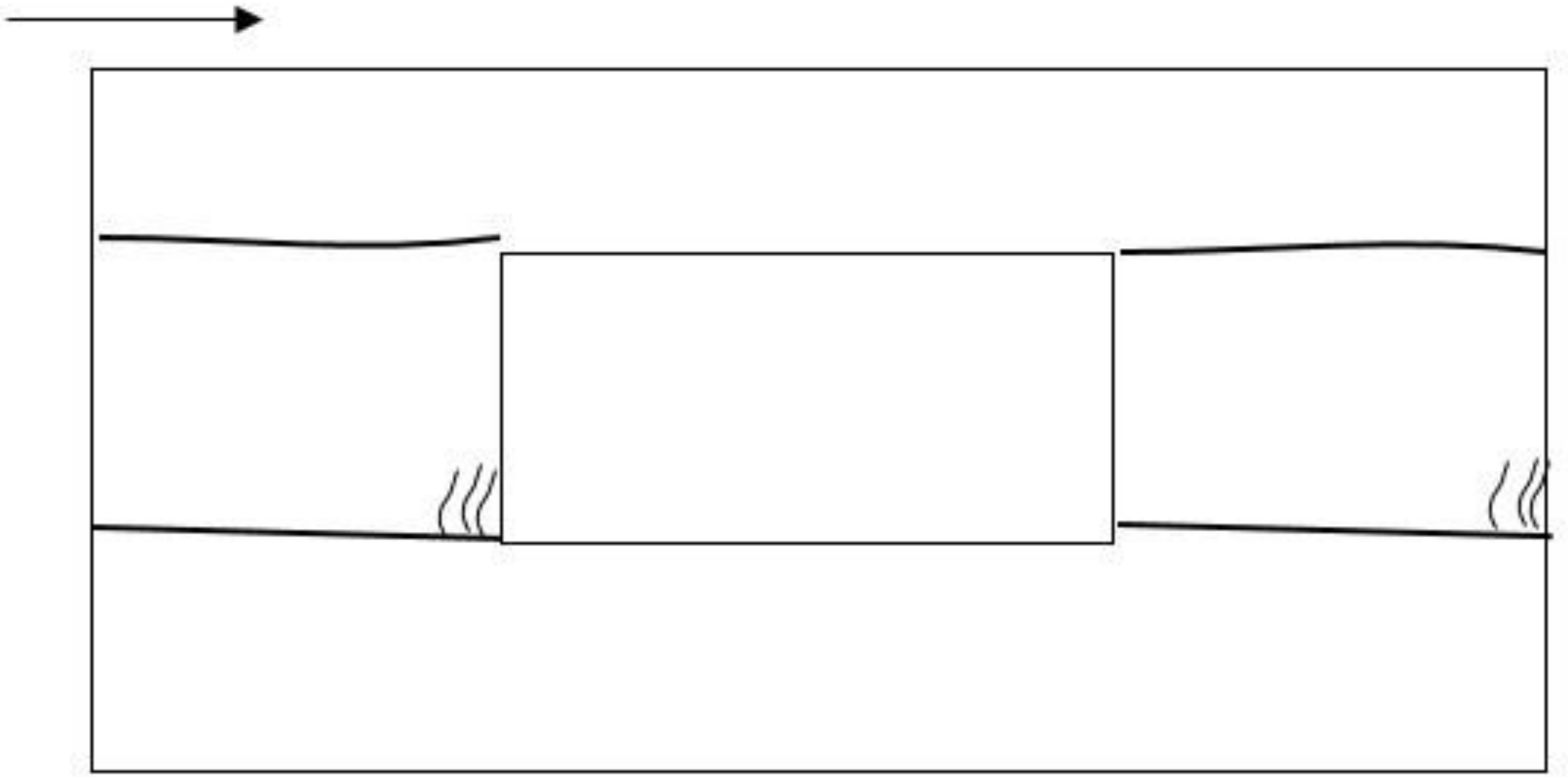


IMAGE 26 – 150 or 200 mm Reinforced Masonry Wall Piers
Failure Mode: In-plane bending vertical toe splitting cracking and top and bottom horizontal cracking
Cause: Inadequate vertical reinforcement; poor unit, joint mortar, and grout compressive strength.

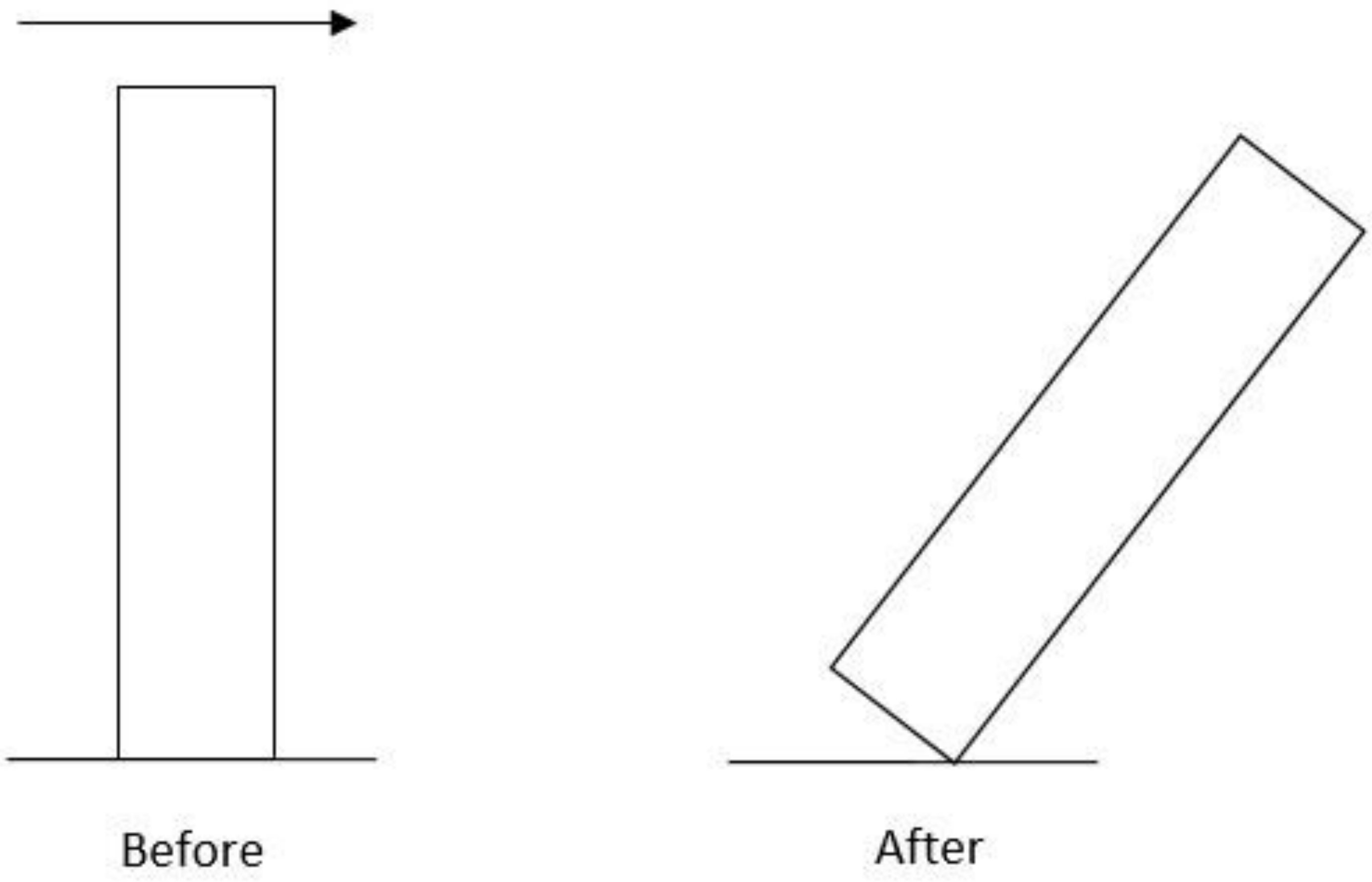


IMAGE 27 – 100 mm Unreinforced Masonry Wall Piers
Failure Mode: Out-of-plane Toppling Instability
Cause: Slenderness too high; bearing stress too low

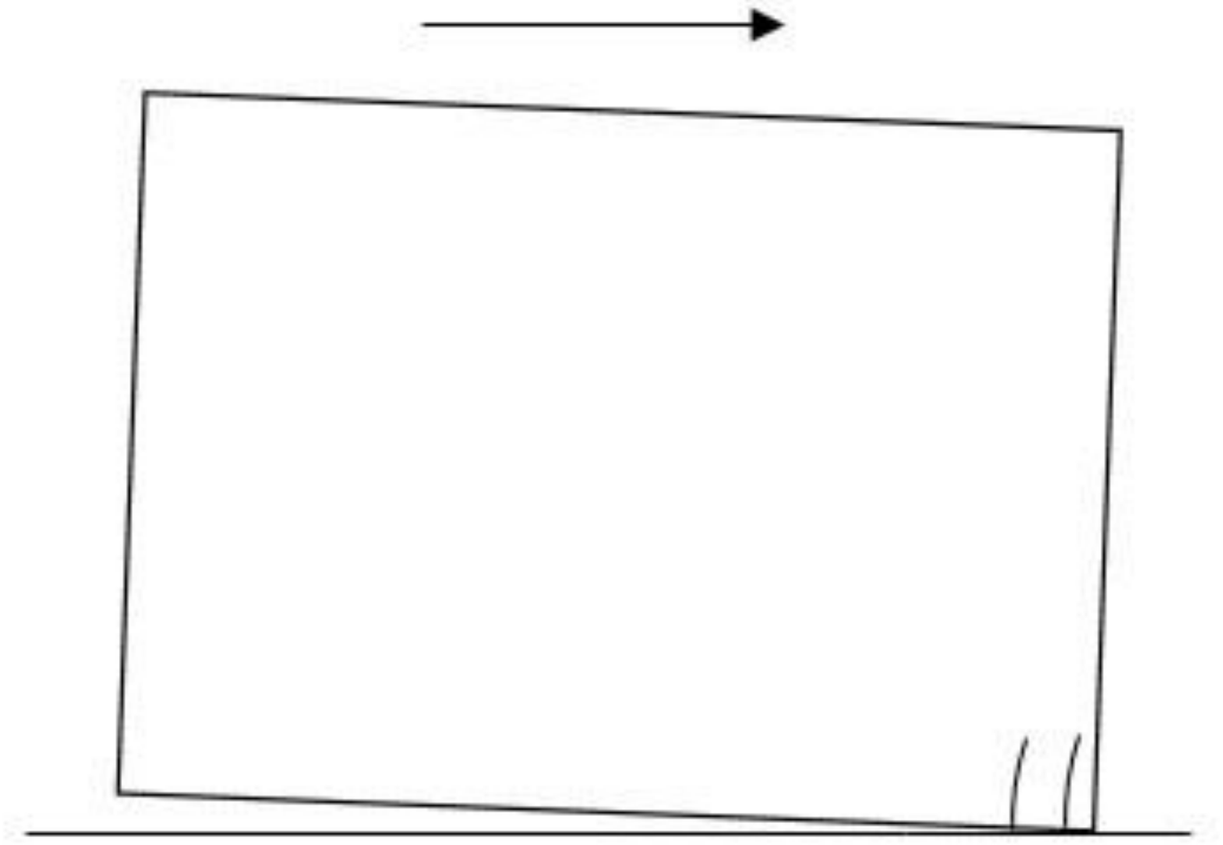


IMAGE 28 – 100 mm Unreinforced Masonry Wall Piers
Failure Mode: In-plane sliding-vertical toe splitting cracking
Cause: Poor unit, or joint mortar, compressive strength; bearing stress too low

CARIBBEAN HOUSING EVALUATION RESULTS - TWO-STOREY HOUSES: FAILURE MODES/CAUSES

IMAGE 29 – EXAMPLE OF 2-STOREY HOUSING

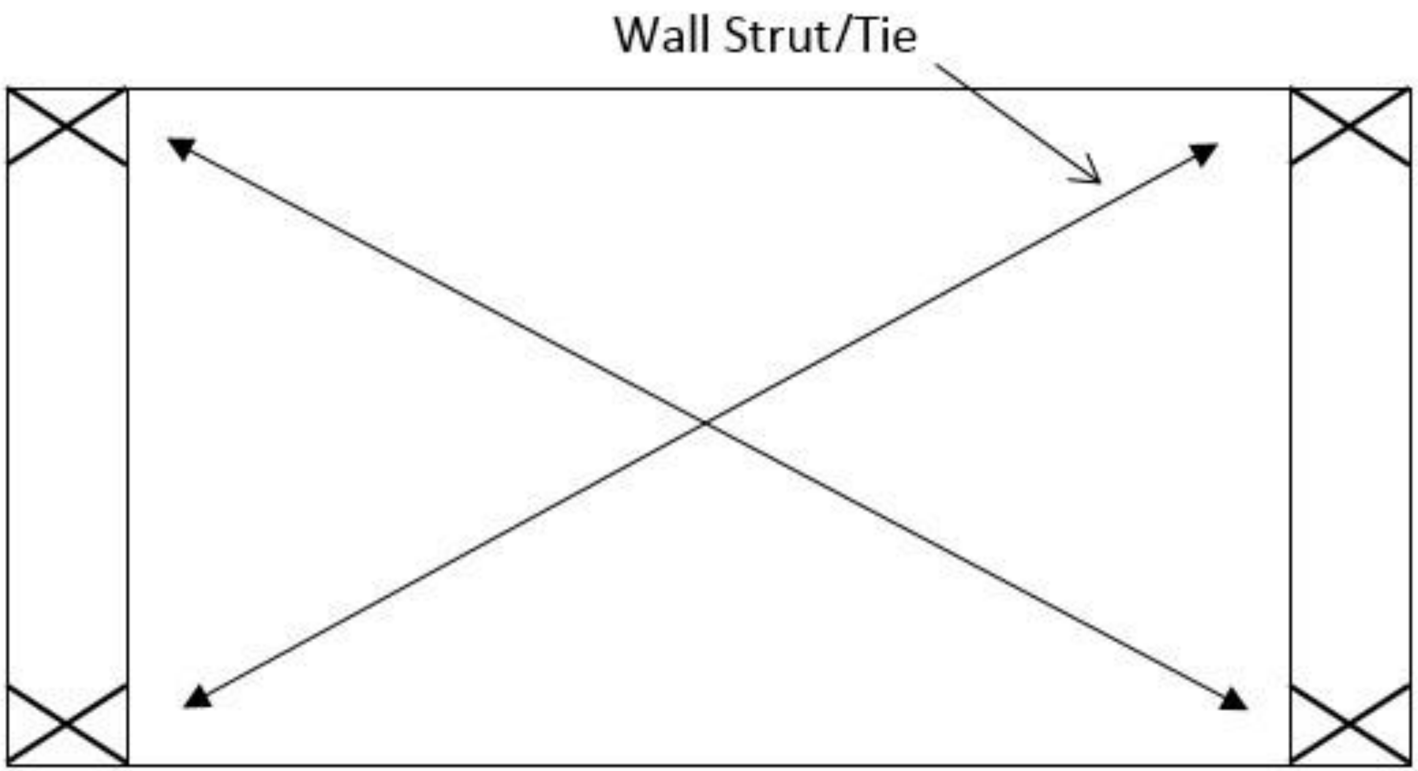


IMAGE 30 – 150 mm RC Masonry Infilled Frame (No Openings)
Failure Mode: Masonry diagonal cracking and RC columns shear failure
Cause of Masonry Failure: Inadequate horizontal reinforcement; poor unit, joint mortar, and grout compressive strength; possibly excessive vertical rebar
Cause of RC Columns failure: Inadequate or insufficient transverse rebar legs

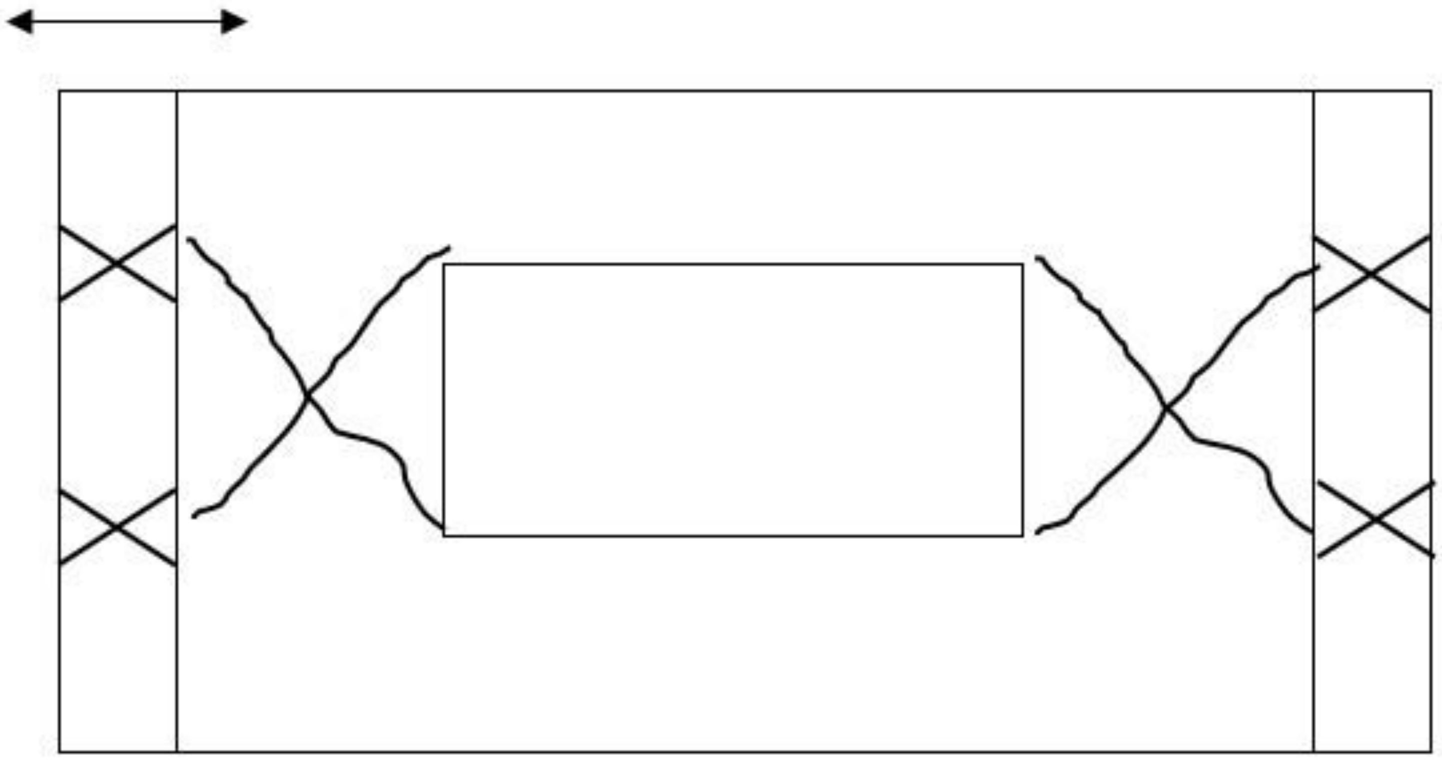


IMAGE 31 – 150 mm RC Masonry Infilled Frame (With Openings)
Failure Mode: Masonry diagonal cracking and RC columns shear failure
Cause of Masonry Failure: Inadequate horizontal reinforcement; poor unit, joint mortar, or grout compressive strength; possibly excessive vertical rebar
Cause of RC Columns failure: Inadequate or insufficient transverse rebar legs

IMAGE 32 – 150 mm RC Masonry Infilled Frame Failure #1 (Dominica Earthquake of 21 November 2004; M,R = 7.0, 27 km; Fling-type Motion)

IMAGE 33 – 150 mm RC Masonry Infilled Frame Failure #2 (Dominica Earthquake of 21 November 2004; M,R = 7.0, 27 km; Fling-type Motion)

CARIBBEAN HOUSING – CHARACTERISTICS OF PROPER STRUCTURAL SYSTEM SOLUTIONS

The impact of Hurricane Maria is such that many owners of proprietary structural systems will be approaching governmental institutions to accept their systems for use. It is therefore very important that any systems not immediately recognized by the appropriate building codes be proven by meeting the Acceptance Criteria of said codes. Furthermore, in a region significantly prone to both hurricane and earthquake hazards, it is the latter that controls the acceptance.

Any proposed alternative seismic structural system solution must be based on appropriate cyclic testing and analysis as required by the seismic codes of practice. However for alternative systems, the most recent seismic codes of practice (ASCE 7-16 and ASCE 41-17) are founded on the *performance-based design* paradigm which in addition to prescribing certain testing and data reduction protocols, state acceptance criteria in consideration of quantified uncertainty (aleatory and epistemic) hence probability of failure based on nonlinear dynamic analysis. Since these codes were only just released, it is likely the data proving acceptability of many proprietary systems to these codes, does not yet exist.

RETROFIT SOLUTIONS FOR EXISTING HOUSING – TYPICAL SOLUTIONS

Typical known retrofit solutions for existing Caribbean houses are as follows:

1. For masonry infilling RC frames: install movement joints
2. For masonry walls: add external reinforcement in the form of rebar or welded-wire reinforcing (WWR); or add an external overlay of composite material such as FRP or ferrocement.
3. For RC columns: add external reinforcement as rebar for longitudinal and transverse steel as rods or WWR; or composite materials such as FRP.

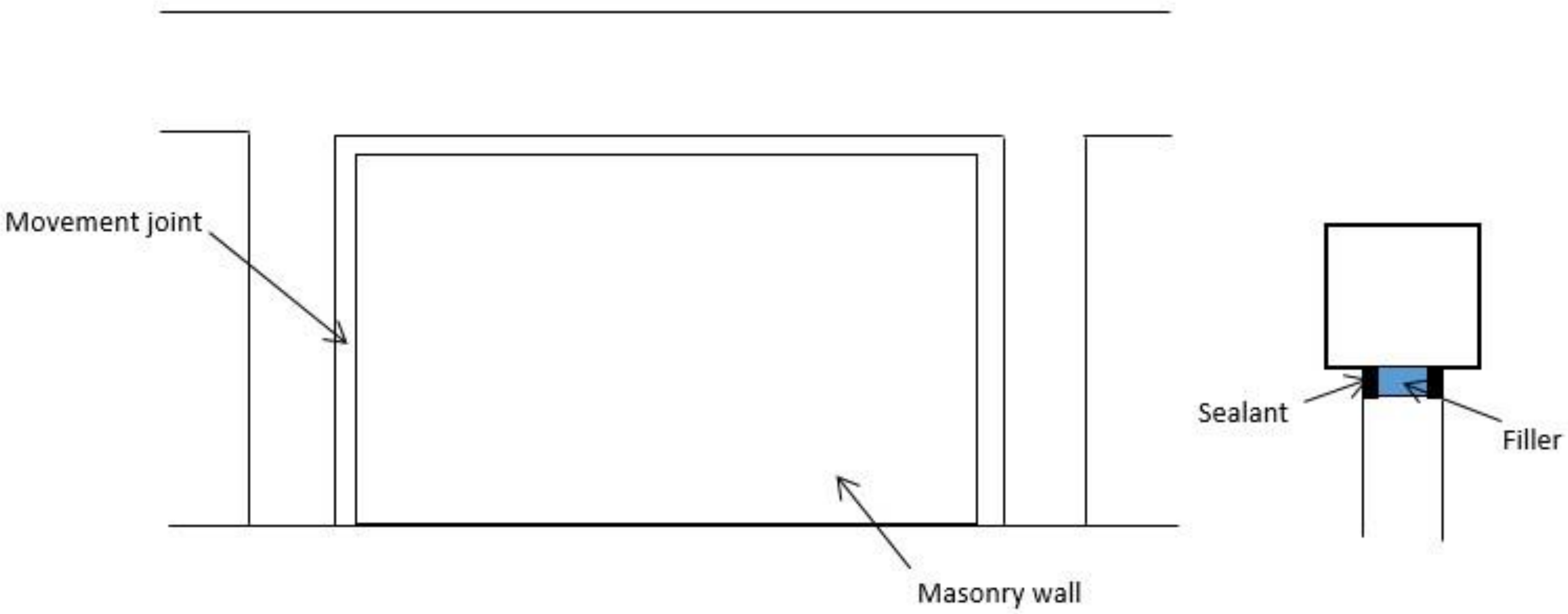


IMAGE 34 – NEW MOVEMENT JOINT

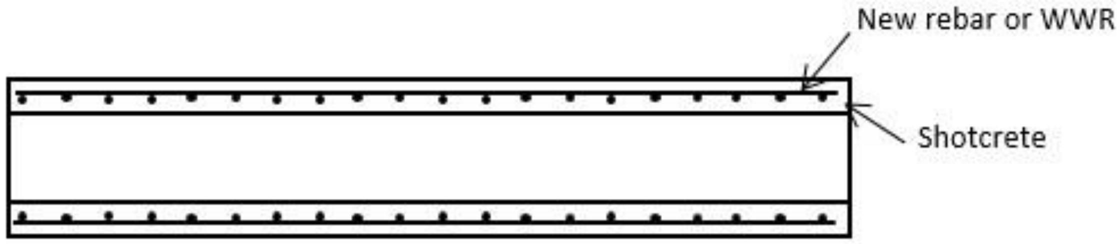


IMAGE 35 – NEW EXTERNAL REBAR OR WWR

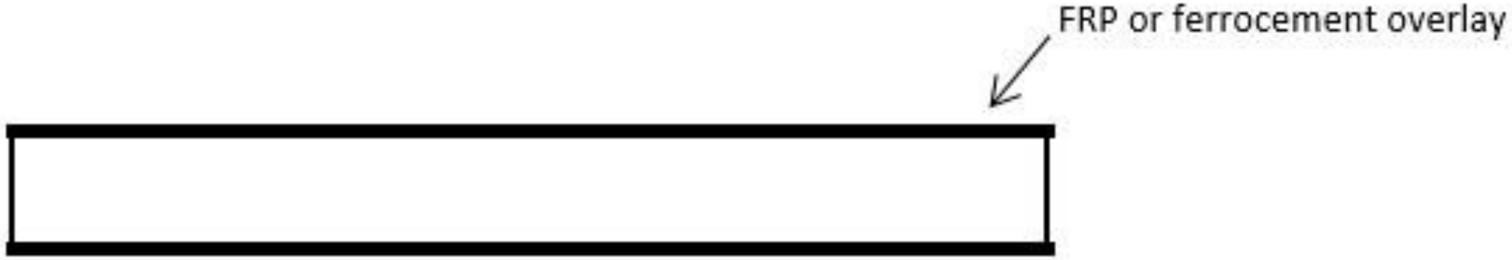


IMAGE 36 – NEW EXTERNAL OVERLAY OF FRP OR FERROCEMENT

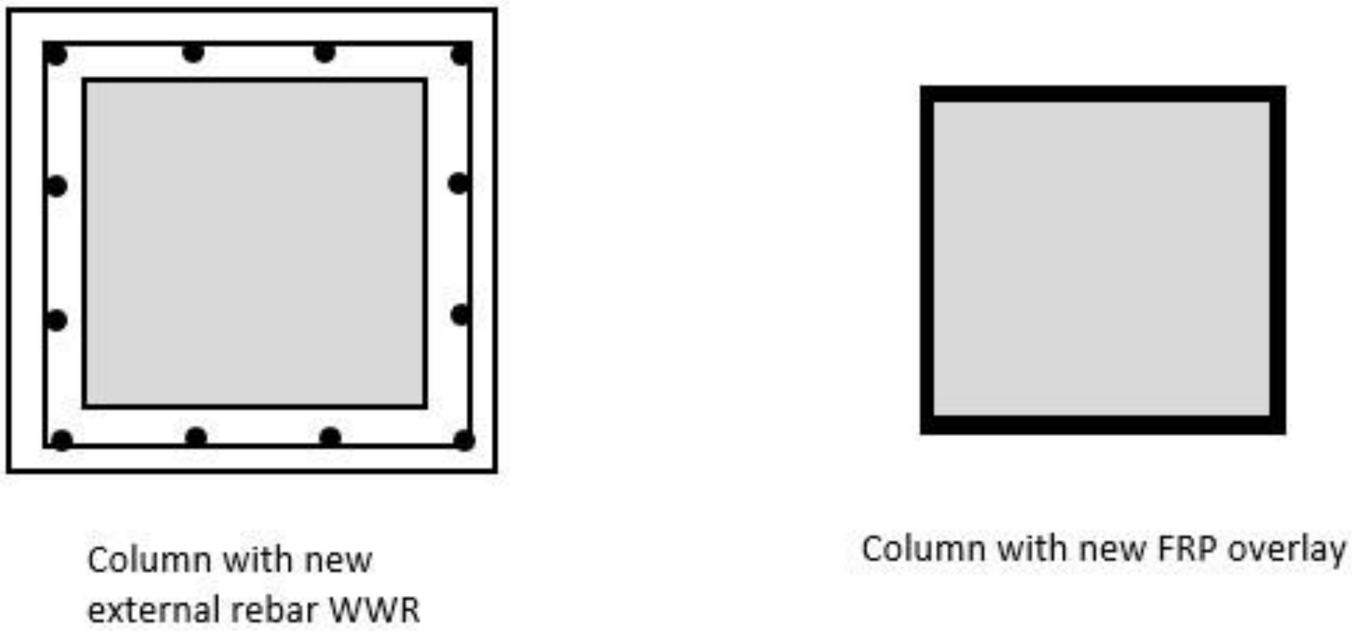


IMAGE 37 – TYPICAL RC COLUMN RETROFIT

RETROFIT SOLUTIONS FOR EXISTING HOUSING – R&D SOLUTIONS

Ferrocement can also be used as a masonry wall overlay in lieu of FRP. At UWI via R&D research, a ferrocement earthquake retrofit solution was derived especially designed for the low bearing stresses of Caribbean construction and can be applied as a community, self-help, or DIY project thus drastically reducing cost. A construction manual is freely available (http://sta.uwi.edu/eng/civil/).

IMAGE 38 – INSTALLATION OF UWI R&D MASONRY WALL RETROFIT TECHNOLOGY

SOLUTIONS FOR NEW HOUSING – TYPICAL SOLUTIONS

The solutions for new housing are whatever is prescribed in the building codes typically used in the Caribbean for housing. However, it is important to note that not all such codes cater appropriately for earthquake resistance. As shown previously, a principal deficiency for reinforced masonry walls is the lack of adequate shear strength due to the lack of suitable and suitably arranged horizontal reinforcement.

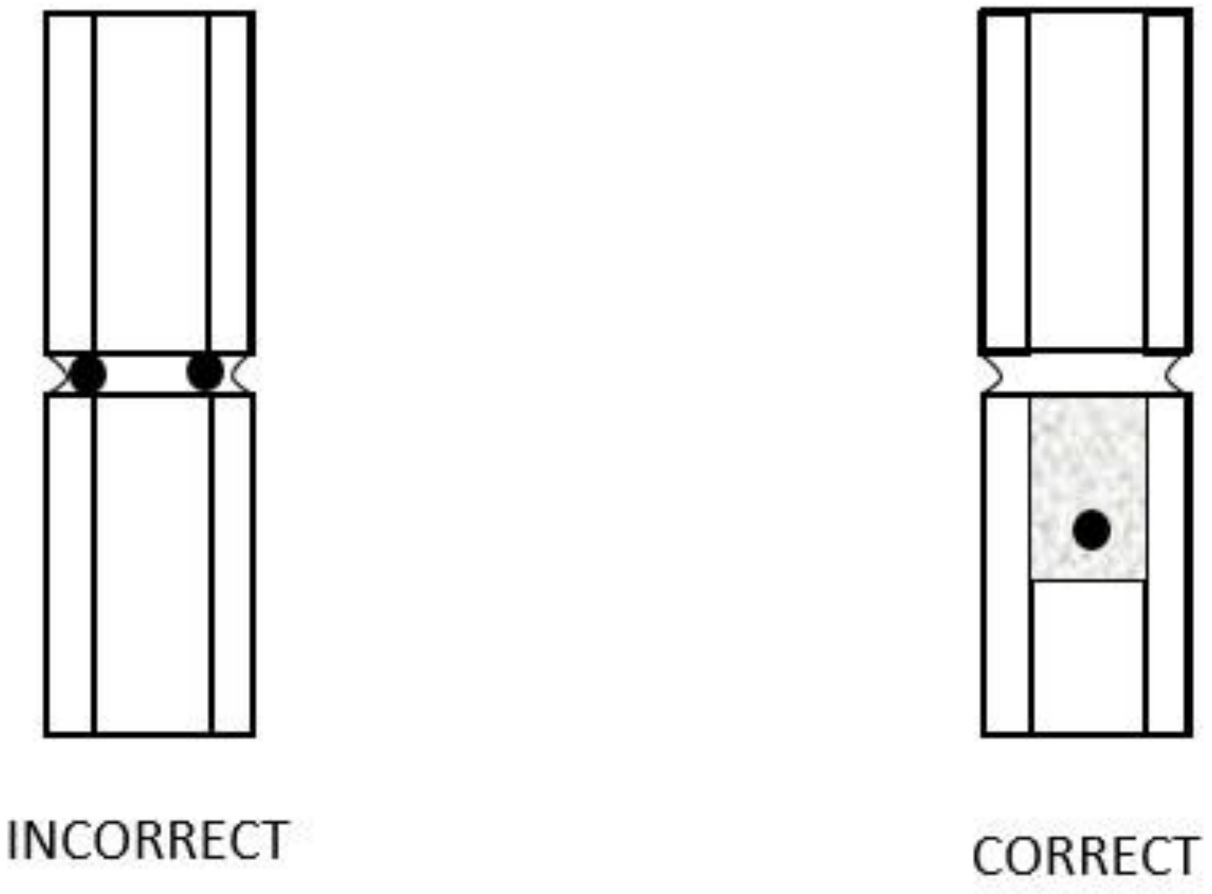


IMAGE 39 – CORRECT HORIZONTAL REBAR PLACEMENT

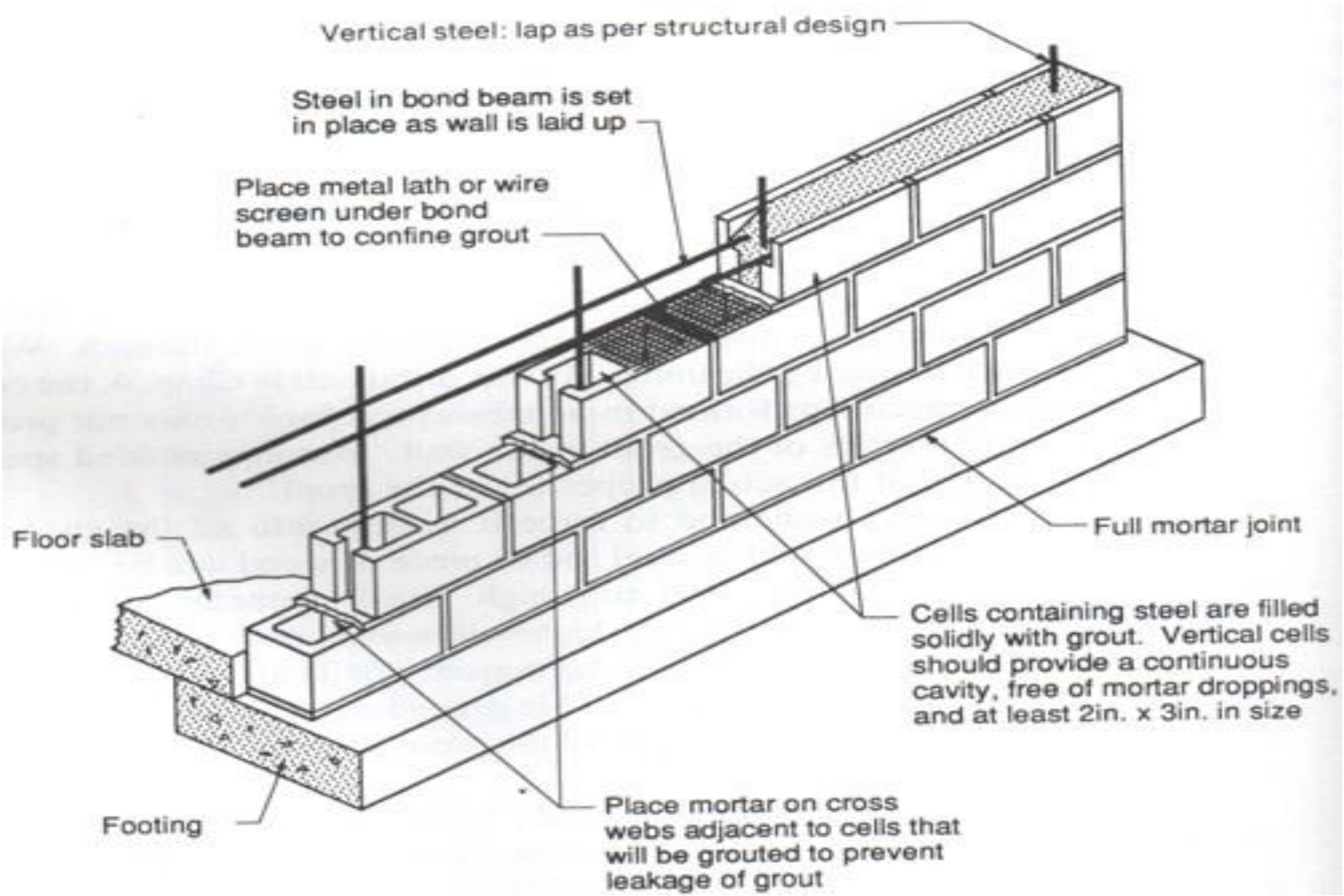


IMAGE 40 – CORRECT SEISMIC MASONRY REBAR PLACEMENT (Credit: Reinforced Masonry Design by Schneider and Dickey)

SOLUTIONS FOR NEW HOUSING – R&D SOLUTIONS

The main shortcoming of the existing systems is that there are too many non-redundant elements on the load path from sheeting to foundation, and that these elements require the input of skilled labor and supervision. Hence new system solutions are required that have the characteristics of: (1) less elements, and (2) high technical element capacities with very minimal need for technically skilled supervision. This will also help in drastically reducing cost by promoting self-help construction.

At UWI the following modular roofing and wall system called "high energy composite" or HEC was derived. HEC is a higher evolution from ferrocement because a plate theory solution was derived for layer-by-layer design with varying matrix types, varying mesh types in each direction, and varying fiber orientations. The system is hurricane and earthquake resistant.

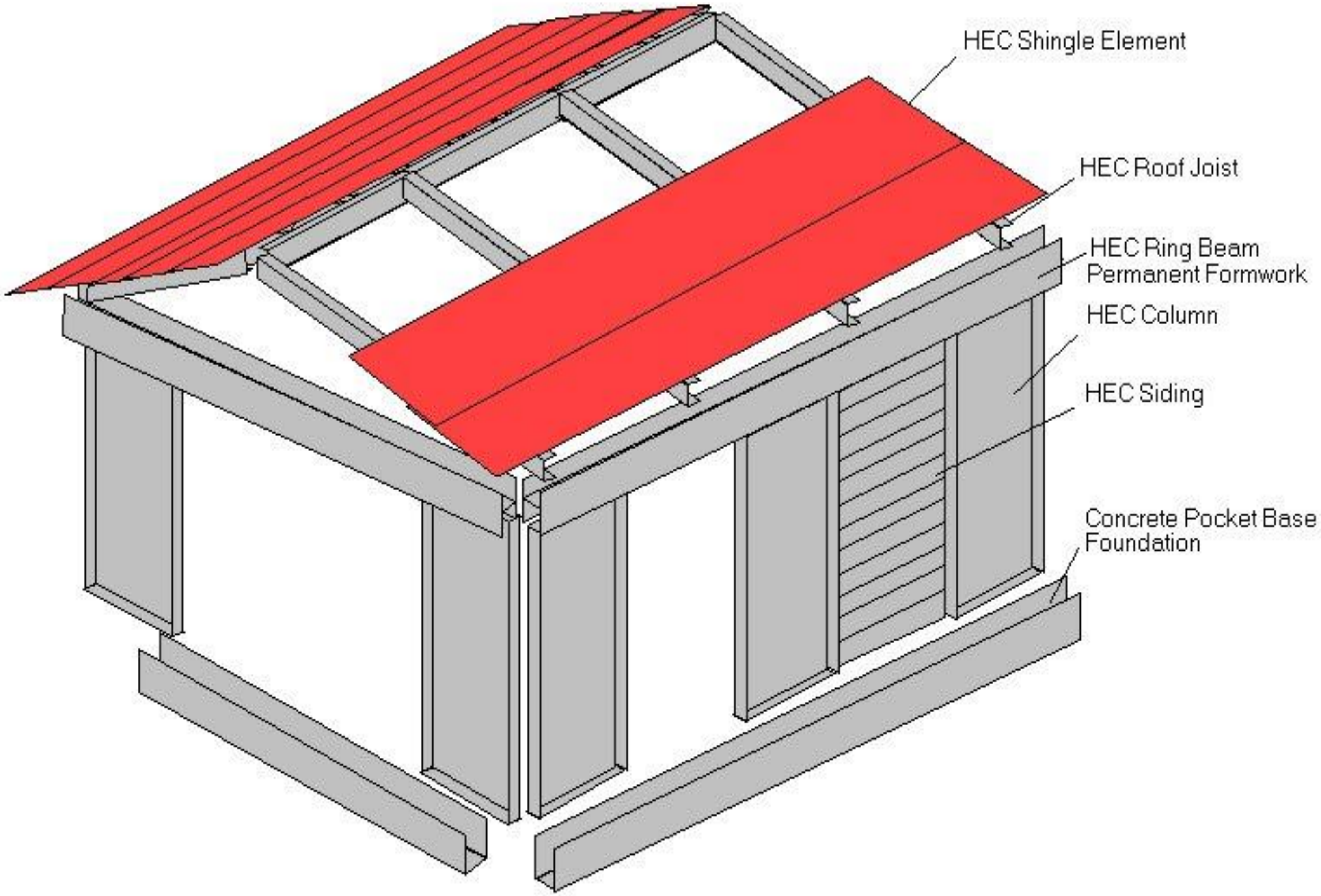


IMAGE 41 – CONCEPTUAL DESIGN OF A HEC HOUSE

ENGINEERED BUILDINGS
HURRICANE MARIA DAMAGE ASSESSMENT

As shown in the following photos, the engineered buildings of Dominica are typically RC moment frames infilled with masonry blocks and under the high wind speeds of hurricane Maria, these performed reasonably well. Damage is concentrated at the roof area when timber roofing is used as opposed to RC slabs, and to glazing and doors, when shutters were not used. This supports the hypothesis that the observed damage to the unengineered structures has poverty at its root cause since it is more expensive to strictly adhere to the building codes.

IMAGE 42 –DAMAGE TO DOWNTOWN STRUCTURES #1

IMAGE 43 –DAMAGE TO DOWNTOWN STRUCTURES #2

IMAGE 44 –DAMAGE TO DOWNTOWN STRUCTURES #3

IMAGE 45 –DAMAGE TO HARBOUR-FRONT #1

IMAGE 46 –DAMAGE TO HARBOUR-FRONT #2 (GARRAWAY AND FORT YOUNG)

IMAGE 47–DAMAGE TO HARBOUR-FRONT #3

IMAGE 48 –DAMAGE TO HARBOUR-FRONT #4 (FERRY TERMINAL)

IMAGE 49 –DAMAGE TO HARBOUR-FRONT #5 (MARKET)

However, there was one major exception to the overall reasonably good performance of the engineered buildings. In this case, the structure is a 6-storey traditional rectangular steel-framed building. *This is characterized by moment frames having the columns' major-axes oriented in one direction, and masonry infilling the moment frames, but without movement joints.*

The following photos show the damage that occurred and which was localized in the stair-well area of each floor. There are horizontal cracks along the three segments of each of the stairwell core masonry walls at each storey of the structure. The cracks are located at a level in line with the soffit of the floor beams.

IMAGE 50 - TYPICAL CRACKING OF STAIRWELL CORE WALLS

IMAGE 51 - TYPICAL CRACKING OF STAIRWELL CORE WALLS (OTHER SEGMENTS)

IMAGE 52 - CRACKING AT WALL-COLUMN INTERFACE

DOMINICA ENGINEERED STRUCTURES - FAILURE MODES/CAUSES

For the previous case, the cause of the cracking is the horizontal force exerted by the hurricane, through the floor slab, and supporting beams. This is shown in the sketch below for two floors. The cracks may also have been initiated by previous hurricane or earthquake events but exacerbated by the present hurricane event. The horizontal force is an in-plane shear force that mobilizes flexural, shear, and axial stresses within the wall that interact with the surrounding frame. In this case, it seems that the interface joint of the wall with the frame forms a weak plane so the cracking takes place at those locations.

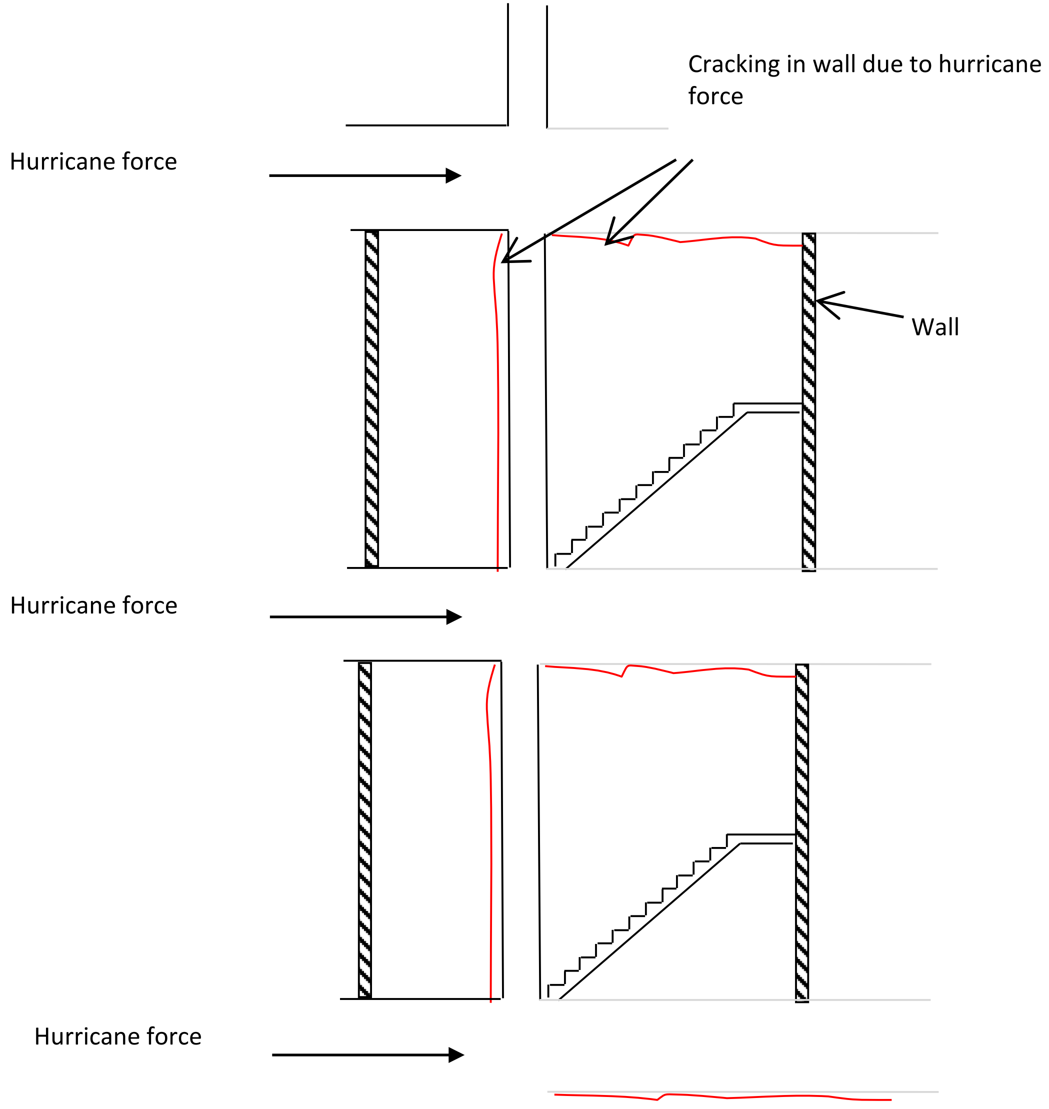


IMAGE 53 – CRACK FORMATION IN IN-FILLED MASONRY WALLS

The cracked walls probably do not impair the present ability of the structure to address future hurricanes in terms of the overall building performance though this should be verified by further analysis.

Experience may have suggested that the structural design would be adequate for the anticipated wind load. However, it is possible that the actual load was greater than recommended by present building codes due to global warming.

As stated earlier for the case of unengineered buildings, a FRP or ferrocement overlay can be applied as a retrofit solution for the masonry walls.

However, in terms of resistance to future earthquakes, the overall system characteristics for this steel-framed building are not consistent with Best Practice, and this may be to a significant degree.

EVALUATION OF EXISTING CARIBBEAN ENGINEERED BUILDINGS

Throughout the Caribbean, for engineered buildings the forms of construction are similar to what was observed in Dominica. Given the hurricane performance of the Dominica engineered buildings and overall observation of the structural concepts employed, Caribbean engineered buildings are generally expected to perform reasonably well under future hurricanes. However as stated earlier, the need to re-evaluate the design wind speeds for the Caribbean due to global warming will necessitate a re-evaluation of specific buildings.

One area that may be vulnerable is the case when unreinforced or lightly reinforced masonry is used for the building's envelope walls in framed buildings. Given the high variation in the compressive strengths of masonry units in the Caribbean and the sporadic supervision during construction of these walls, they may be in need of retrofitting. The use of overlays of FRP or ferrocement is applicable, as discussed previously for the case of unengineered buildings.

One area that may be vulnerable is the case when unreinforced or lightly reinforced masonry is used for the building's envelope walls in framed buildings. Given the high variation in the compressive strengths of masonry units in the Caribbean and the sporadic supervision during construction of these walls, they may be in need of retrofitting. The use of overlays of FRP or ferrocement is applicable, as discussed previously for the case of unengineered buildings.

However, as regards the likely performance of engineered Caribbean buildings <u>under earthquakes</u>, this is a different matter for steel-framed buildings. For RC-framed buildings built before about 1980, performance is expected to be similar to the case of two-storey houses as previously discussed. For RC-framed buildings built since about 1980, performance is expected to be relatively better since the hinging regions are expected to be better reinforced. However since movement joints are typically not used, the effect of the masonry infills may reduce the overall performance to below intended levels and they may need to be retrofitted.

As noted in the previous section, traditional steel moment-framed buildings are not consistent with international Best Practice for seismic resistance. Traditional steel moment-framed framed buildings are characterized by moment frames having the columns' major-axes oriented in one direction, and masonry infilling the moment frames, but without movement joints. These frames have bolted end plate moment-resisting connections in the column major or strong-axis direction but without consideration of the dynamic stresses induced, and use of the wrong type of weld. The beams and columns sections are typically seismically non-compact. In the column minor or weak-axis direction, the typical connection also uses an end plate but bolted directly to the column’s web. These connections are also frequently modelled incorrectly. Given the relative severity of this case, steel moment-framed buildings are the focus of the remainder of this presentation.

CARIBBEAN ENGINEERED BUILDINGS EVALUATION RESULTS - FAILURE MODES/CAUSES

The seismic deficiencies of Caribbean steel moment-framed buildings will cause flexural failure in the beams or columns without stable hinging, hence building collapse by insufficient energy absorption. Or, failure by yielding and buckling of the column panel zones leading to frame instability hence building collapse. There will be frame hence building collapse by the formation of storey mechanisms in the columns minor-axis directions. If there are masonry infills, failure in this manner will also occur but after failure of the masonry walls which may also damage the columns before failure as described previously.

Other common deficiencies are: lack of lateral bracing of seismic main beams in the direction of the floor decking secondary beams, and lack of diaphragm chord reinforcement in floor slabs of precast joists or precast panels.

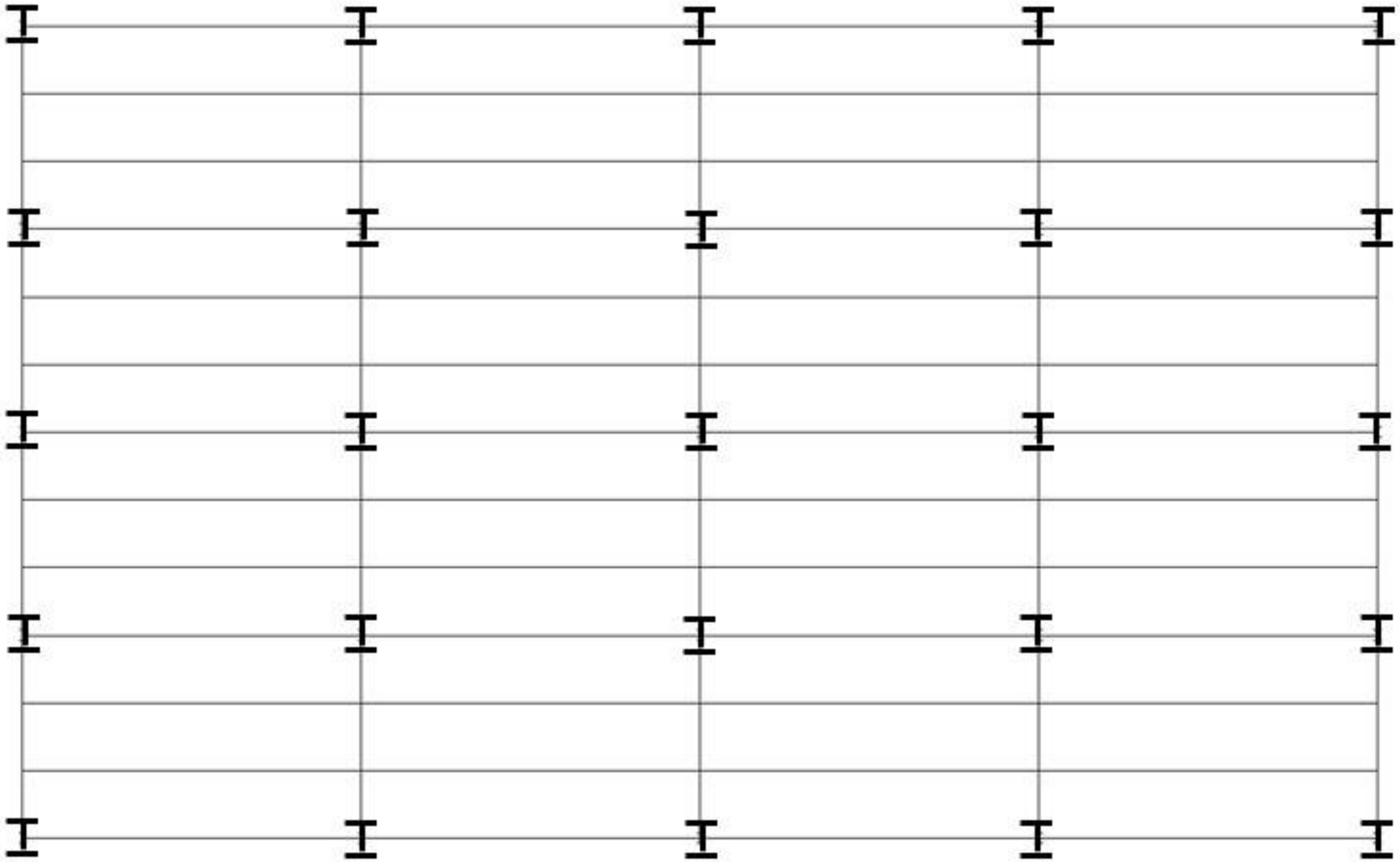
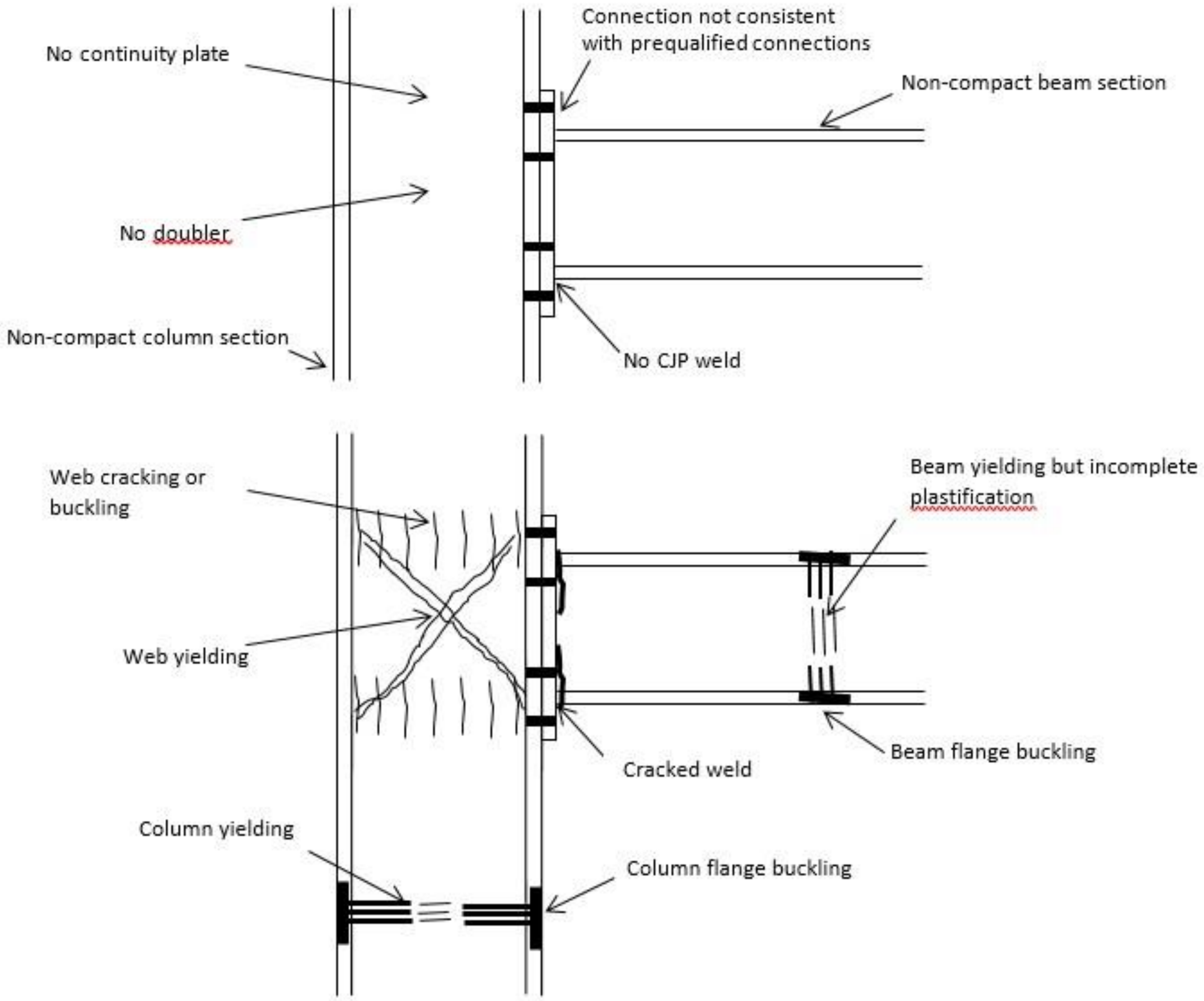
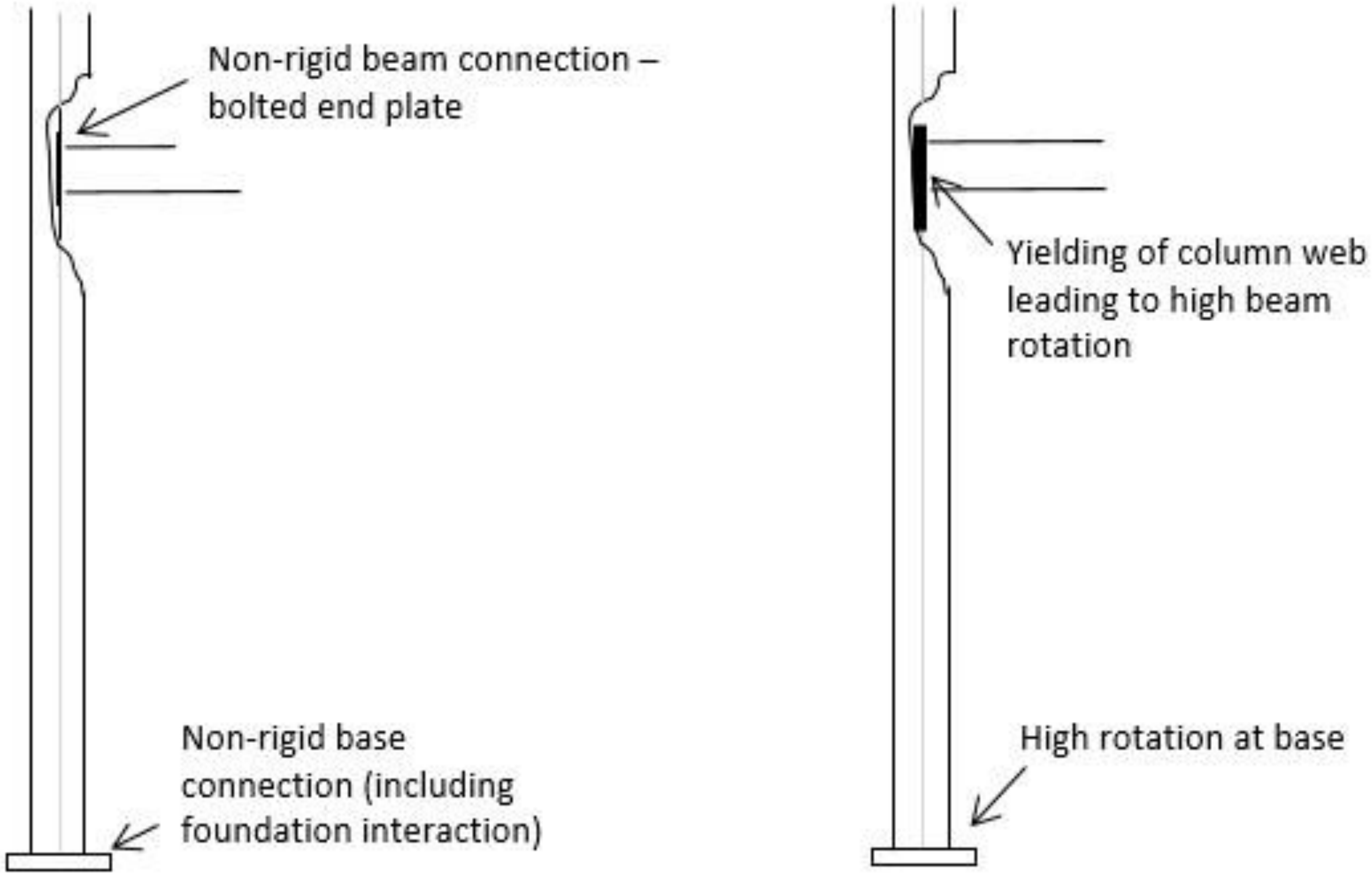


IMAGE 54 – TRADITIONAL CARIBBEAN STEEL MOMENT-FRAME CONCEPTUAL DESIGN

IMAGE 55 – Typical frame in column major axis direction
Failure Modes: Panel zone shear or tension yielding or buckling; connection weld cracking; beam yielding or column yielding; beam or column local buckling
Causes: Column web too thin or material too weak; weld too weak; incomplete beam plastification due to slab interaction or excessive unsupported length; beam or column flanges or web too thin





TYPICAL CONSTRUCTION IN MINOR AXIS DIRECTION

RESPONSE IN MINOR AXIS DIRECTION LEADING TO STOREY MECHANISM HENCE COLLAPSE

IMAGE 56 – TYPICAL MINOR-AXIS CONNECTION

RETROFIT SOLUTIONS FOR EXISTING ENGINEERED BUILDINGS – TYPICAL SOLUTIONS

Typical retrofit solutions for steel moment-frame Caribbean buildings are as follows.

For frames in the major-axis direction:

i. Beams and columns can be strengthened by adding external steel cover plates
ii. Bolt forces can be reduced by adding haunches to the ends of beams
iii. Panel zones can be retrofitted by adding doubler plates by extending the end plates of beams transverse to the joint, and adding continuity plates
iv. Certain bents can be braced by installing new steel braces, or new reinforced masonry or RC shear walls can be added; access to the foundation for installing transfer components may be a limiting factor
v. Existing masonry within the frame can be utilized to convert the system from a steel moment frame to steel masonry infilled frame. The same considerations apply as discussed for RC masonry infilled frames for housing, and the capacity of the existing foundation will need to be checked or this approach is not viable
vi. New bents can also be added which can be braced or have RC shear walls

For frames in the minor-axis direction, since there is no such thing as a standard prequalified seismic steel minor-axis moment-frame connection:

i. Certain bents will have to be braced or reinforced masonry or RC shear walls added; access to the foundation for installing transfer components may be a limiting factor

ii. New external frames or shear walls must be added

RETROFIT SOLUTIONS FOR EXISTING ENGINEERED STEEL MOMENT-FRAMED BUILDINGS – R&D SOLUTIONS

A R&D retrofit solution is one that must be verified by appropriate testing and in a configuration that replicates the field conditions of the existing frame. AISC 341 specifies the testing for general qualification of connections (Appendix P) and for project-specific qualification (Appendix S).

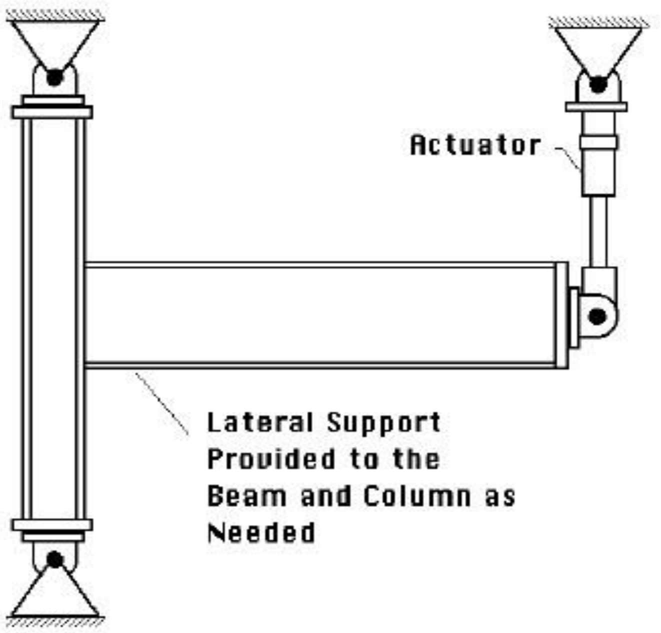


IMAGE 57 – A CONNECTION TEST SET-UP (*Credit: FEMA 350*)

A connection is deemed acceptable if upon application of a specific cyclic loading protocol: (1) the connection is capable of sustaining an interstory drift angle of at least 0.04 radians and (2) the measured flexural resistance of the connection, determined at the column face, is at least $0.80M_p$ of the connected beam at an interstory drift angle of 0.04 radians.

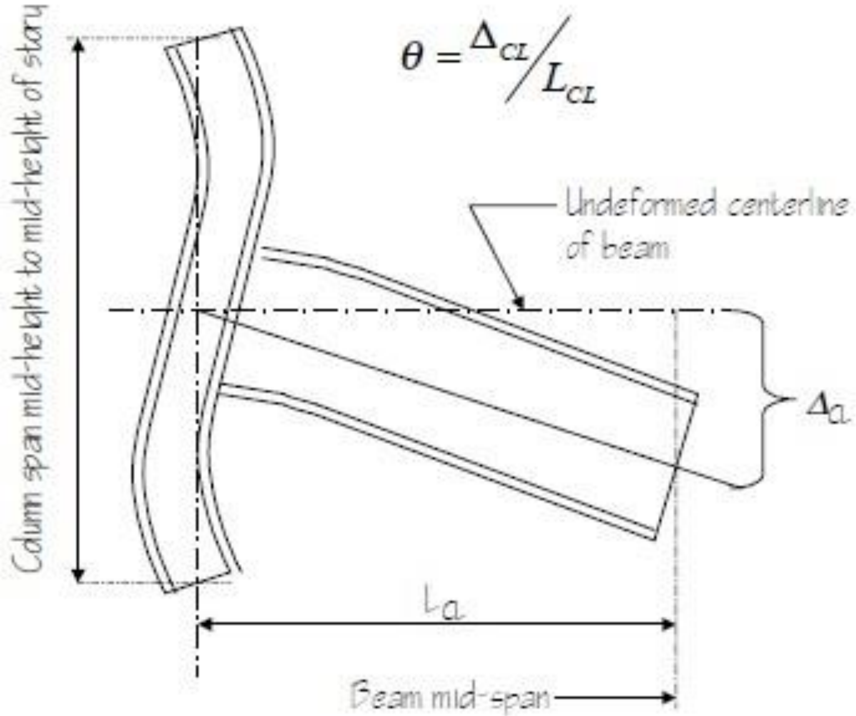


IMAGE 58 – DEFINITION OF INTERSTORY DRIFT ANGLE (*Credit: FEMA 350*)

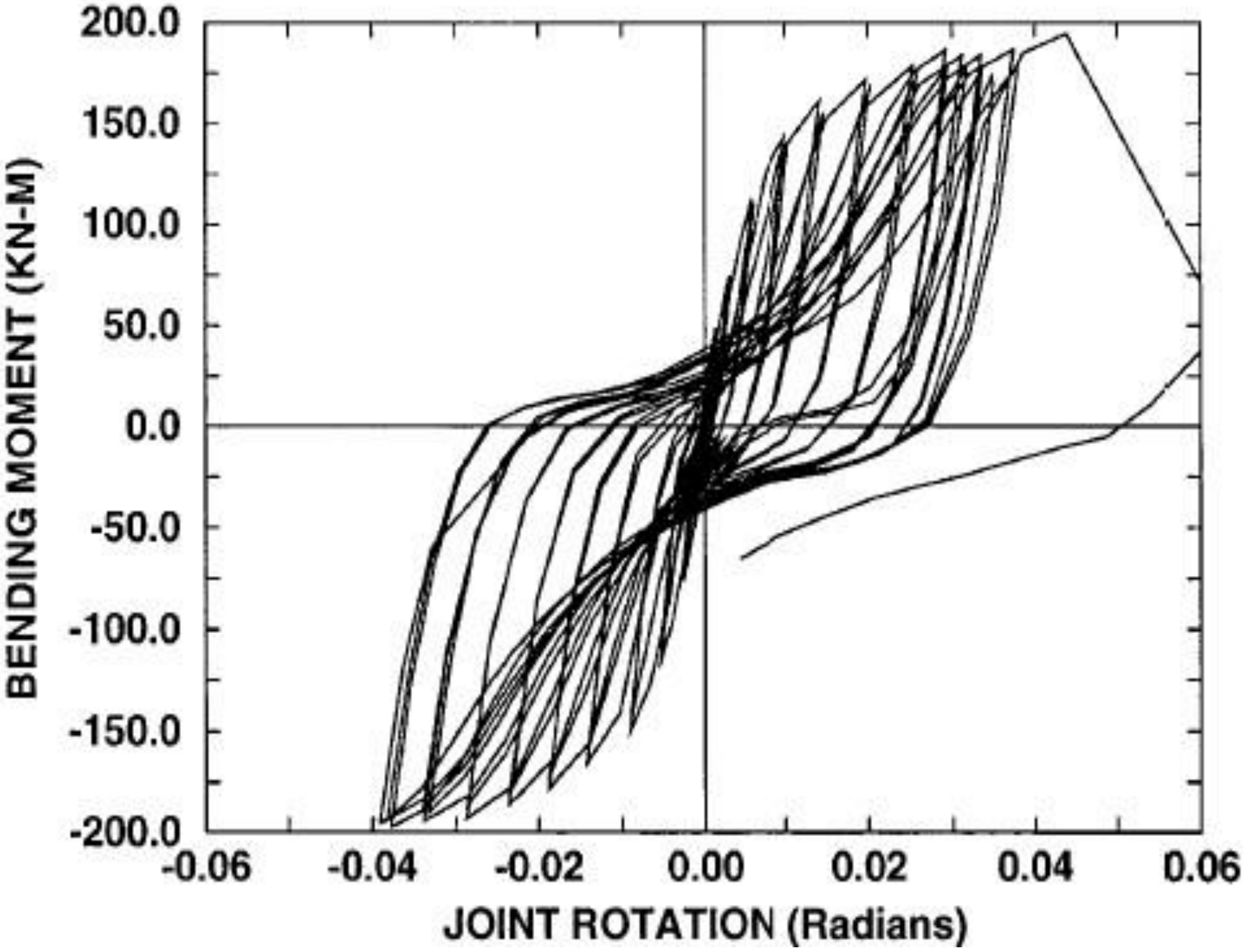


IMAGE 59 – GOOD TEST RESULTS (*Credit: FEMA 355D*)

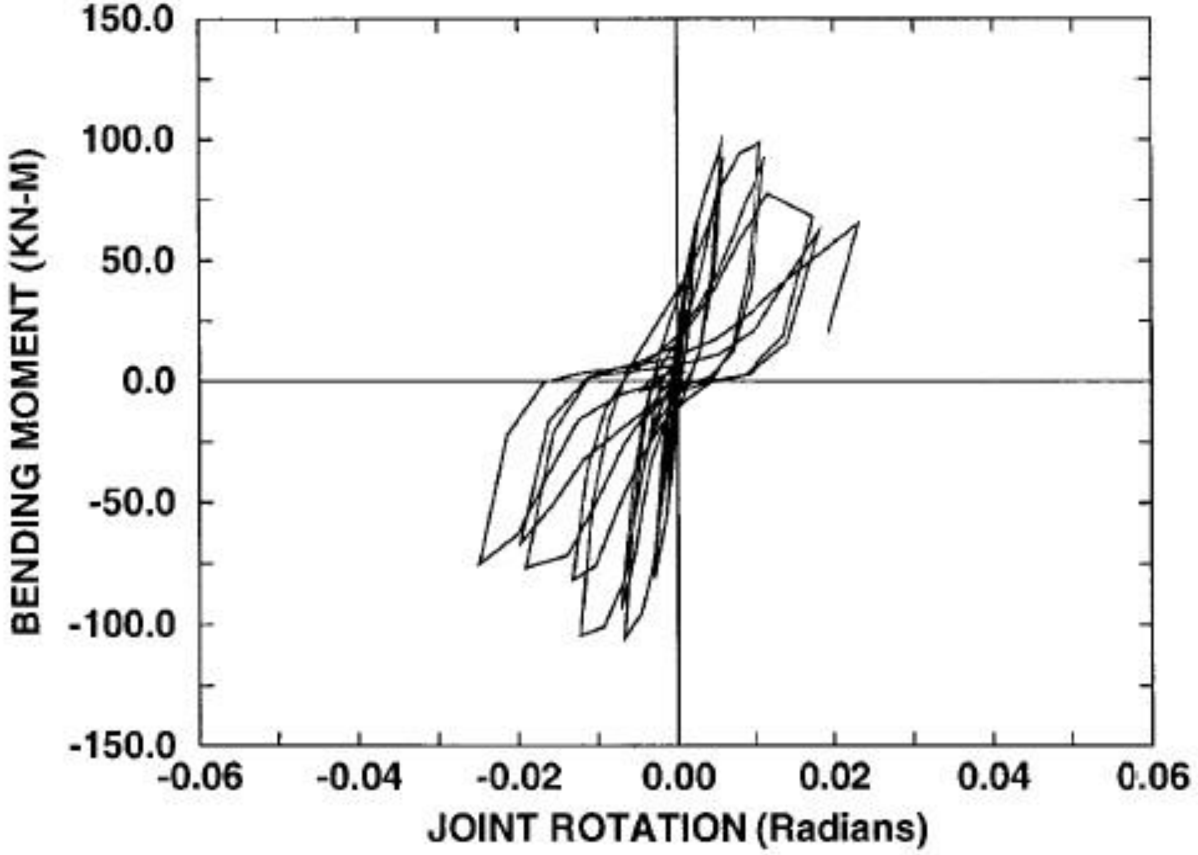


IMAGE 60 – POOR TEST RESULTS (*Credit: FEMA 355D*)

In the Caribbean, since the tradition is to use a bolted end plate beam connected to the column web, it may be more convenient to use such a configuration to provide seismic resistance rather than using one of the solutions for the minor-axis direction as stated in the previous section. Since such a connection is not prequalified research is required. The sketch below shows the conceptual design of one such connection.

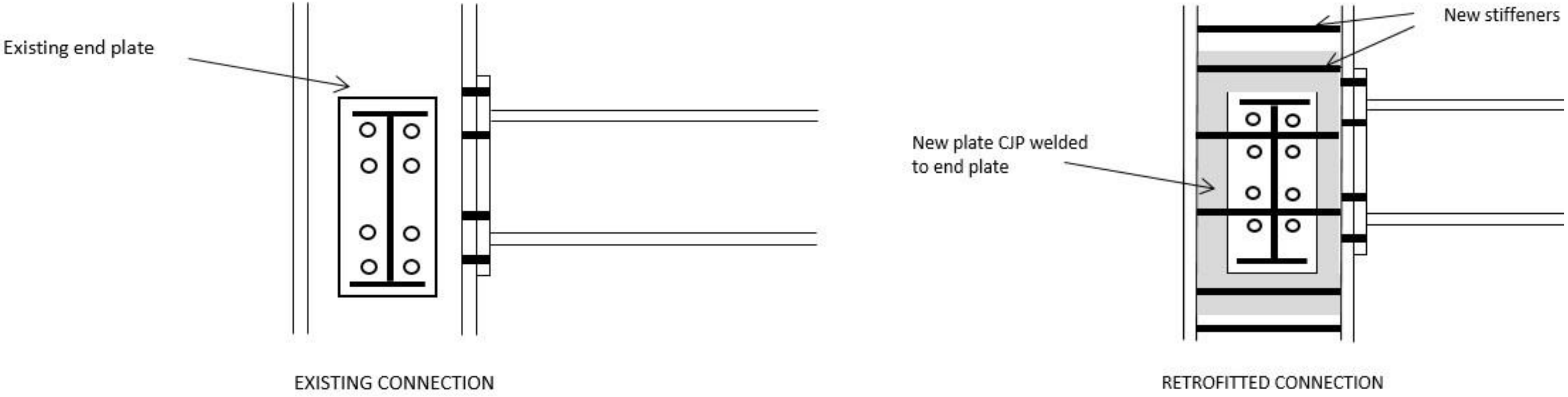


IMAGE 61 – CONCEPTUAL SEISMIC RETROFIT DESIGN FOR MINOR-AXIS CONNECTION

SOLUTIONS FOR NEW ENGINEERED BUILDINGS – TYPICAL SOLUTIONS

The typical solutions for earthquake resistant framed buildings as presented in the design texts and guides are based on U.S.A traditions in which case braced frames or shear walls are used in the columns' minor-axis direction.

Hence in the Caribbean, in which case architects seem to prefer moment frames in both directions, it has become customary to "turn" certain columns to provide major-axis frames in both directions. Another Caribbean practice for moment frames is to use the frames on all the grid lines to provide seismic resistance, which is uneconomical. The following sketch shows how a more economical arrangement can be achieved, using the layout of IMAGE 54. Note that for the non-seismic frames, the codes require that they be designed for deformation compatibility.

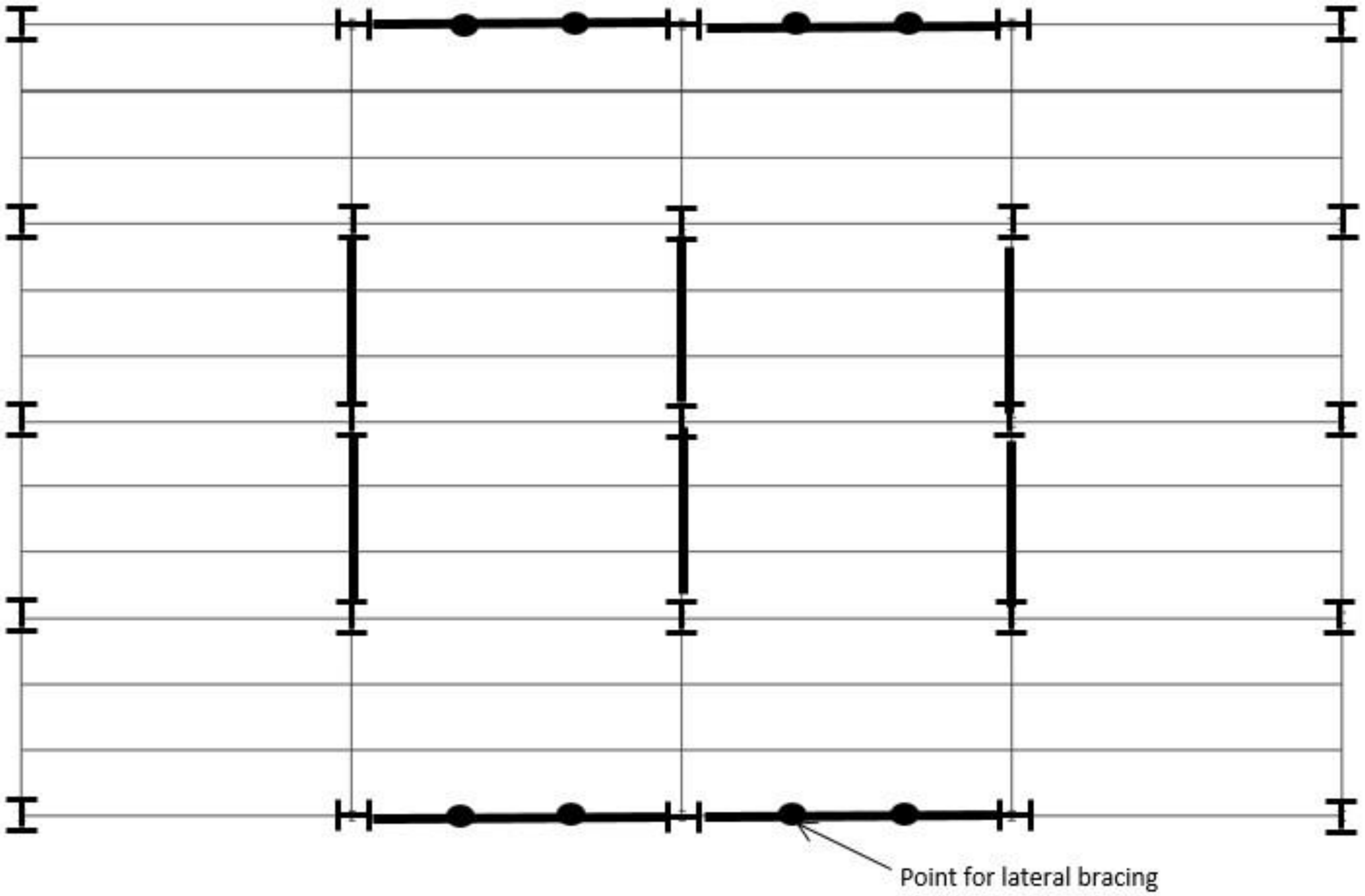


IMAGE 62 – CONCEPTUAL SEISMIC MOMENT FRAME LAYOUT FOR A NEW BUILDING

Also for the non-seismic frames, typical U.S.A practice is to use a shear tab connection at the beam-to-column joint. Since this is a very flexible connection it is presumed that this enables the full seismic storey shear to the transferred to the seismic frames, and that the energy induced in the non-seismic frames by the deformation of the seismic system is acceptable. It is also a simpler connection hence reduces the cost of the non-seismic frames.

However, the author prefers another approach for the non-seismic frames for the Caribbean based on constructability and data scarcity considerations. The reasons for this are firstly, unlike for the case of prequalified connections within seismic frames, as far as the author is aware there is no specification for the approval testing for deformation compatibility of non-seismic joints hence though allowed in the U.S. codes, a rigorous basis is lacking. Secondly, the level of QA/QC of fabrication practice in the Caribbean is quite low relative to the U.S.A where the codes are derived. In the U.S.A the design recommendations are tied to construction and testing practices and appropriate checks and balances are generally in place to ensure compliance with the design. Hence reliance on a very flexible beam-to-column joint together with a flexible base joint is deemed risky in the low QA/QC environment of the Caribbean.

Hence the author prefers the following solutions for the non-seismic frames. For the beam-to-column flange, use the typical extended end plate connection we are accustomed to. It is not necessary that this be the pre-qualified 4E, 4ES, or 8ES pre-qualified connections. For the beam-to-column web connection, the AISC Driscoll connection (1983) recommended by the AISC Steel Construction Manual (13 th ed. pp 12-15) is preferred. Alternatively, if higher ductility demand is expected, the Astaneh-Asl (1995) connection may be preferred. As these connections are stiffer they add a measure of redundancy hence reduced drift demand in an environment where scant attention is paid to nonstructural elements, and the recommended design accelerations are based on a relatively small data set (i.e. 22 records) implying significant uncertainty in the hazard estimation.

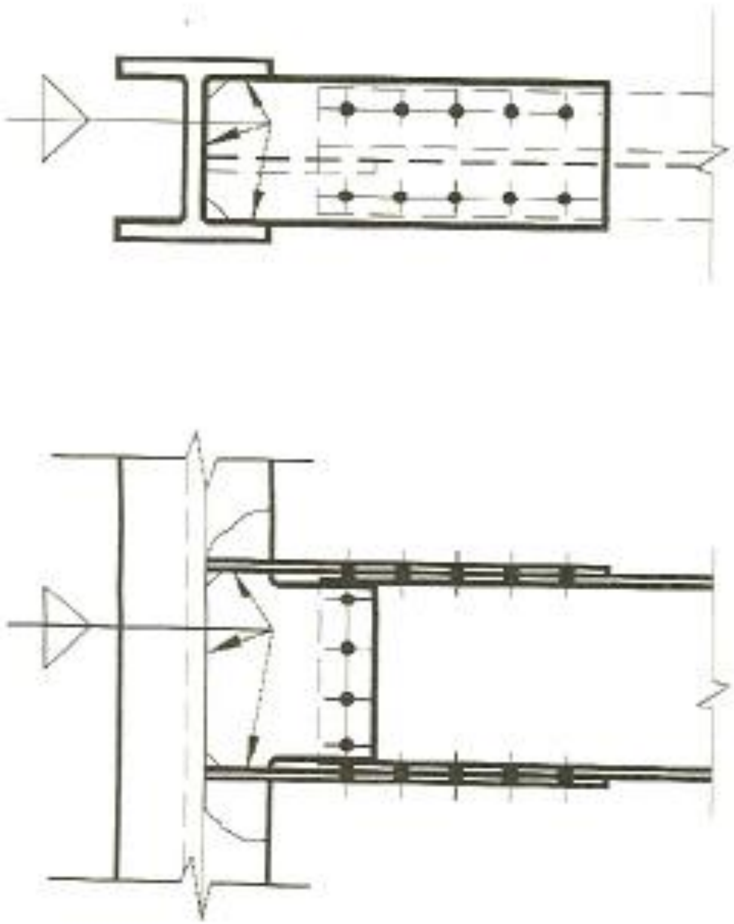


IMAGE 63 – THE DRISCOLL CONNECTION (*Credit: AISC Steel Construction Manual, 13 ed*)

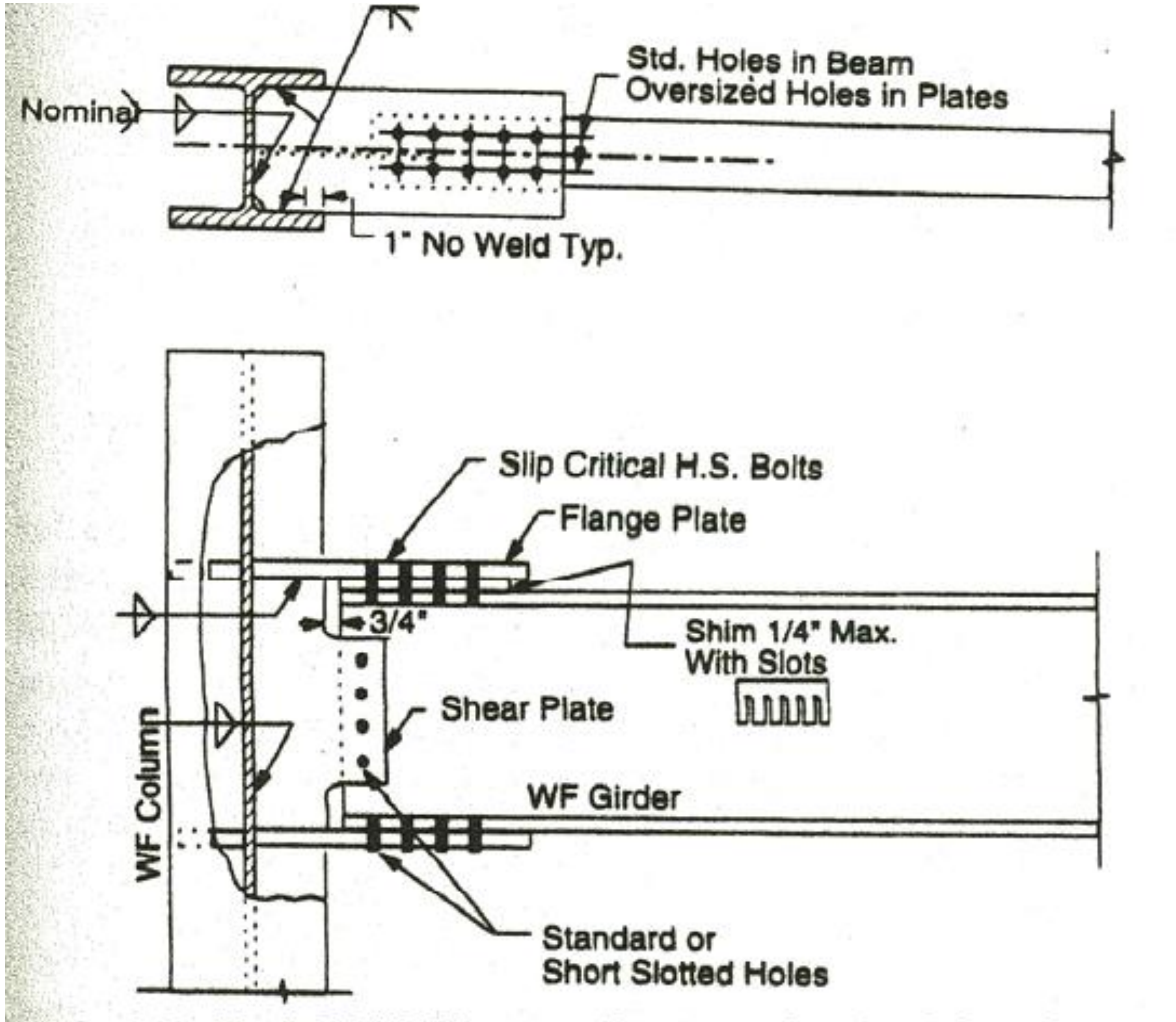


Figure 4.10 Typical CW-BB connection to weak axis of the column (Astaneh-Asl, 1995).

IMAGE 64 – THE ASTANEH-ASL CONNECTION

SOLUTIONS FOR NEW ENGINEERED BUILDINGS – R&D SOLUTIONS

At the department of Civil and Environmental Engineering, UWI, there are no R&D solutions for new seismic steel buildings in progress at this time. Given the aforementioned particular characteristics of the Caribbean environment, the following possible research projects seem reasonable to pursue:

1. The intense research effort by the SAC (SEAOC-ATC-CUREE) after the Northridge 1994 earthquake is the basis for the seismic steel provisions of the present. During that research effort for moment resisting frames it was decided to only provide systems for the column major or strong-axis direction. However cyclic tests were done for the minor-axis direction and it was concluded that minor-axis solutions are possible, but the research was not continued. The proposed UWI research is therefore to expand the work done by Engelhardt et al (2000), Gilton et al (2000), etc. Note that these configurations are similar to those of Driscoll and Astaneh-Asl. However, the new cyclic testing requirements and consideration of epistemic and aleatory uncertainty, implies that any previous research will need to be re-examined.

2. The same R&D concept presented in IMAGE 61 for retrofitting applications, can also be used for new construction.

BARRIERS TO IMPLEMENTATION OF SOLUTIONS

LACK OF ENFORCEMENT

Since most of the Caribbean's building stock (approximately 70 percent) is comprised of unengineered buildings, it is important that codes for housing and other small buildings be mandatory and enforced by the relevant authorities. This means that construction must only be allowed if the design is approved by the state, and the construction must only be approved based on sufficient inspections and materials acceptance testing during construction.

Similarly for engineered or larger buildings, construction must only be allowed if the design is approved by officials of the state who are suitable trained in the building codes and allied documents. The construction must only be approved based on sufficient inspections and materials acceptance testing during construction. Furthermore, one particular requirement that is not being practiced is the "peer review" phase of the design approval process. However if implemented this phase will only be effective if the reviewers have proven competence in the building codes and allied documents.

The legal requirements for making building codes mandatory are surprising absent in certain Caribbean territories such as Trinidad and Tobago.   All of the solutions mentioned above will be impotent if these legal changes are not made.

INADEQUATE TRAINING

The design and construction solutions for Caribbean buildings in terms of acceptable losses due to hurricanes and earthquakes require a large and continuous training effort of its professional engineers. This is because the central topics are outside the core of the knowledge base of typical civil engineering undergraduate degree programs and most graduate programs worldwide, and also not covered by professional certification exams except for certain states of the U.S.A. These topics are: nonlinear cyclic constitutive relations; the Limit Theorems and their application; pushover analysis, nonlinear dynamics, and algorithms for solving systems of nonlinear equations. In addition to these, modern performance-based design requires knowledge of: the Total Probability Theorem; Fragility Analysis, and Statistical Design of Experiments.

In the developed world, proven competence in building structural design, especially in earthquake-prone regions, is acquired by certification. Such certification is based on exams or evaluation of relevant work done and such evaluation is performed by a team of certified individuals in an environment in which a conflict of interest is impossible or highly unlikely. Furthermore, in developed countries, if the individual does not engage in continuous professional development in order to keep abreast of technical developments, he or she may lose their certification and have to reapply.   Since such a certification practice is absent in the Caribbean, this itself is a factor contributing to a non-resilient Caribbean since having a code is useless if it cannot be properly interpreted and used.

THE END

THANK YOU FOR YOUR TIME

(http://sta.uwi.edu/eng/civil/)

Richard P. Clarke

30 January 2018