Shelter 6: Structural Assessment – Haiti

1.1 Introduction and Purpose

Arup was commissioned to carry out a structural review to assess and validate nine selected shelter designs for the IFRC. This document summarises the information gathered for and the key outcomes of the verification of the structural performance of Shelter 6, built by the Spanish Red Cross.

Summary Information:

Location: Haiti, Leogane

Disaster: Earthquake 2010

Materials: Galvanised steel frame, timber studs, plastic sheeting for walls, Aluzinc roof sheeting, concrete foundations, bolts, screws and nails

Material source: Steel frame procured internationally and shipped from Spain, other materials sourced locally and transported by truck

Time to build: 2 days

Anticipated lifespan: 24 months

Construction team: UNKNOWN

Number built: 5100

Approximate material cost per shelter: 1700 CHF (2010)

Approximate programme cost per shelter: 4300 CHF (2010)

Shelter Description:

The shelter consists of a galvanised rectangular steel frame with an 8.5 degree mono-pitch roof and a suspended floor. The height to the eaves is 2.55m and 3m to the ridge and there is no bracing. The shelter is 3 x 6 m on plan and has 6 columns spaced on a 3m grid, fixed to 800x800x400mm rectangular reinforced concrete foundations using a 300x300x6mm base plate and four ordinary bolts per base. The raised floor is also supported by 13 additional stub columns on 100x100x6mm base plates bearing directly on to the soil. The main structure is three primary structural frames spanning in the transverse direction with rectangular hollow section columns.

The roof cladding is corrugated steel sheeting nailed to steel secondary roof members spaced at 0.75m intervals spanning between the three primary frames. Timber studs are screwed to the steel members and the plastic wall sheeting attached to this. Additional timber sub-framing is used to form windows and doors.

The shelter is demountable and can be unbolted and foundation bolts cut to reuse the frame. The concrete foundations are not reusable. The intention is to use the structure in a modular manner, putting two side by side to form a double pitched roof structure or 4 together to use as communal facilities. The frame is durable since the members are galvanised. The plastic sheeting is vulnerable to damage and requires replacement over the intended lifespan.

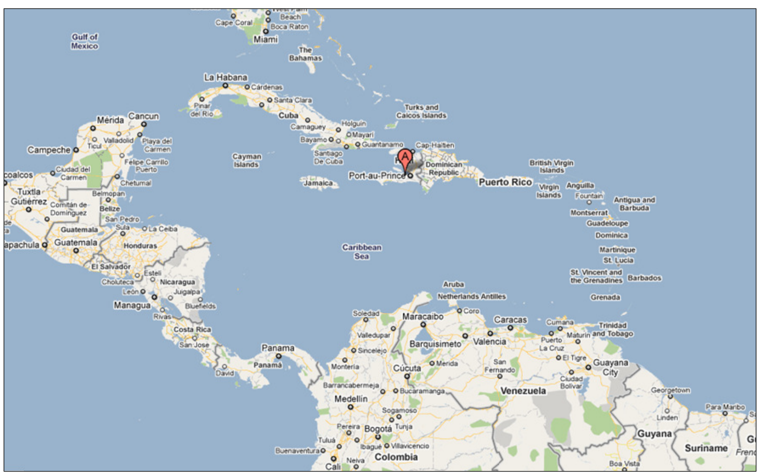
This assessment is based on the input documents listed in Appendix A. From the information provided the Spanish Red Cross Solution manufactured by Castelo has been checked in this case study.

1.2 Location and Geo-hazards

1.2.1 Location of Shelter

Haiti, Island of Hispaniola, Caribbean

The exact location of the shelters in Haiti is unclear but it has been assumed that they have been built near Leogane, the epicentre of the January 2010 earthquake near Port-au-Prince. The shelters have been sited on mountainous and rough terrain.



1.2.2 Hazards

A summary of the natural hazards faced in Haiti is given below[1]:

- HIGH Earthquake Risk. Port-au-Prince is situated between two known fault lines and has a long history of major earthquakes. The US Geological Survey (USGS) with funding from USAID released initial seismic hazard maps for Haiti on 2 April 2010 that give the peak ground accelerations (PGA) expected from ground shaking in Haiti[2]. Based on the latest USGS study for the region a peak ground acceleration of 1.0g has been assumed for an earthquake with a 2475 year return period. This value has been amplified according to the site using the average shear wave velocity over the top 30m of soil.
- HIGH Wind Pressures. Haiti is in a hurricane belt and is prone to tropical cyclones. The shelter was originally designed for wind speeds of 140km/hr but will be checked for a basic wind speed of 217km/hr for a return period of 300 years[3]. See Section 1.8.3 for wind loading details.
- MEDIUM Landslide Risk due to earthquakes or flooding if shelters are located near potentially unstable slopes.
- MEDIUM Flood Risk. High rainfall in May and September may lead to flooding which has been provided against by elevating the floor by 50-65cm.
- Hot humid and tropical climate with temperatures averaging 25-30^{o}C.

1.3 Geometry

The geometry was determined using the drawings provided by the steel fabricator; see Figure 1.1 for the key members and levels.

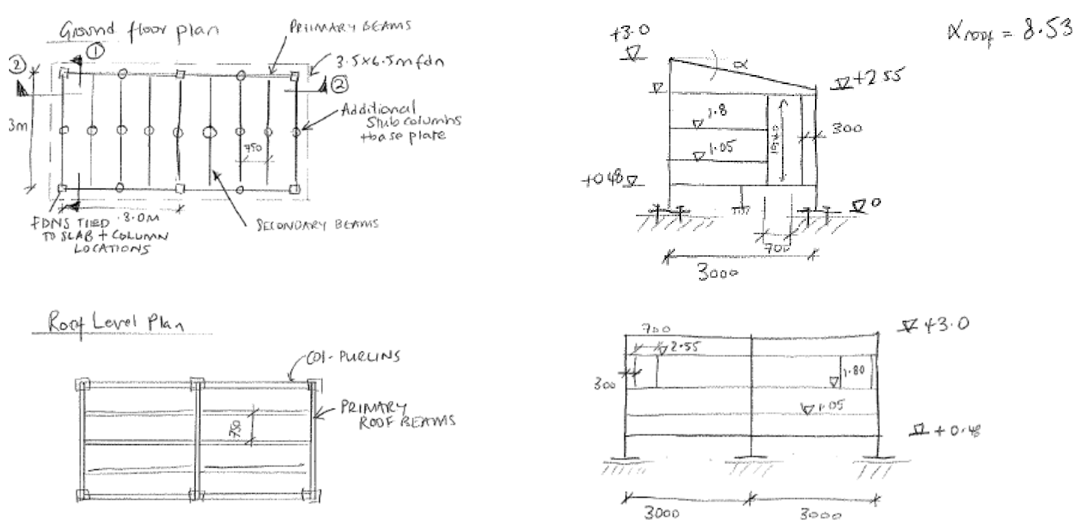


Figure 1.1 – Sketches of Geometry

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1 See Appendix A, Reference 3.
2 See Appendix A, Reference 1.
3 See Appendix A, Reference 2.

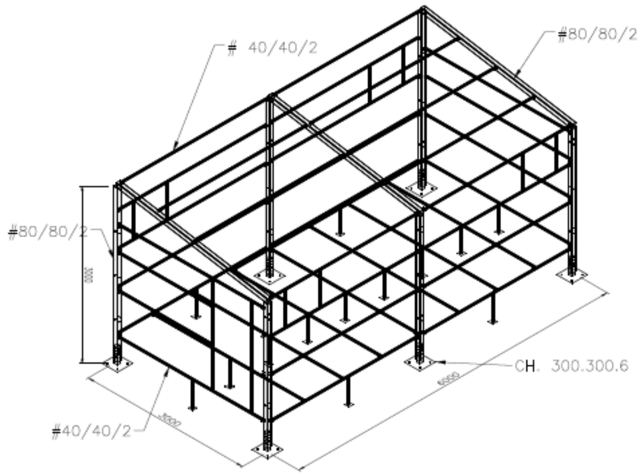


Figure 1.2 – Isometric Drawing of Shelter

The rectangular single storey shelter is 3 x 6 m on plan with a single pitched roof from 2.55m to 3m in height. The shelter has a galvanised steel frame with bolted connections and a nailed corrugated steel sheet roof. A timber sub-frame is screwed to the steel members and acts as support for the plastic wall sheeting only. The shelter has a suspended floor and the foundations are steel column base plates anchored into large cast in-situ square concrete footings, with intermediate stub columns to support the floor.

Modelling and geometry assumptions:

- Shelter geometry is as described, as assumed from the original information and drawings provided.
- All connections are bolted and assumed to act as pinned connections.
- The column base plates are fixed to the concrete using four ordinary bolts. Column stubs have been welded to the base plate and the main column members bolted on to these.
- The timber members are not structural and act as fixings for plastic sheeting and support for doors and windows only. Timber members are fixed to steel frame using 75 x 75mm angles.
- The load on the floor is distributed evenly between the column bases and stub columns.

1.4 Structural System

- Vertical loads are transferred from horizontal beams and purlins back to the six columns which transfer the forces to the ground by bearing of the column base plates onto the concrete foundations and then the soil.
- Resistance to overturning and uplift of the columns is provided by the lesser of the shear resistance of the holding down bolts in the concrete and the weight of the structure and foundations.
- The structure has very little lateral stability since it has been assumed that the connections for each of the three frames are simple connections and provide minimal portal frame action. There is no in plane bracing in the walls or roof and the metal roof decking will not act as a diaphragm, therefore the structure has no real code compliant seismic or wind resistant lateral system (see Figure 1.3).

1.5 Member Sizes

The table below shows the steel frame member sizes that have been assumed for the structural assessment. These sizes have been based on information given in the drawings and Bill of Quantities referenced in Appendix A. The updated Bill of Quantities is given in Appendix B.

Member Ref.	Member Description	Member Size (mm)
P01-06	Columns	RHS 80*80*2
A01	Column Base Plate	Plate 300*300*6
A02	Stub Column Base Plate	Plate 100*100*6
T01	Primary Floor Beams	RHS 40*40*2
T02/04	Secondary Floor Beams	RHS 40*40*2
V01-03	Primary Roof Beams	RHS 80*80*2
T03	Roof Purlins	RHS 40*40*2
T02/03	Wall Transoms	RHS 40*40*2

Currently no hurricane straps are included; possible fixings that could be used as appropriate are shown in Figure 1.3. Connection details are given in Reference 5 and include the use of 2mm thick clamping brackets attached using bolts.

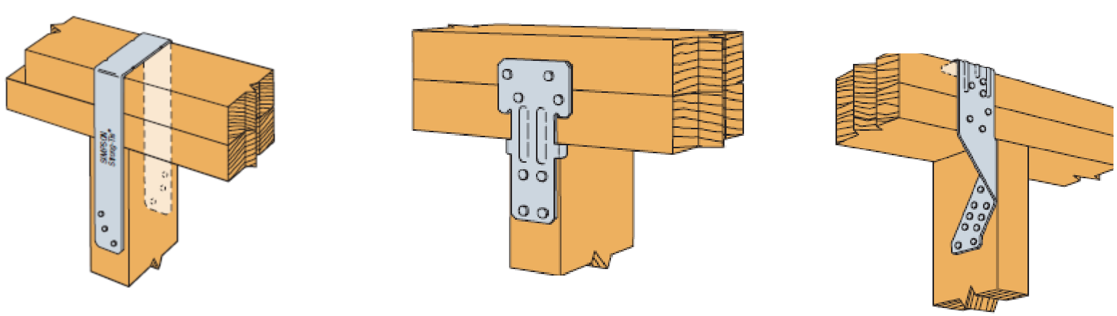


Figure 1.3 – Standard Hurricane Bracket Details

1.6 Materials

Steel frame procured internationally, produced by a Spanish company and shipped to Haiti from Spain. Other materials including timber sourced locally and transported by truck. It is assumed that the steel frame is made from galvanised cold formed steel members, simply connected using M12 bolts and with polythene sheeting walls and a corrugated steel sheet roof.

1.6.1 Material Assumptions

Type	IFRC Specification	Arup Assumption	Comments
Concrete	127.5kg of cement, 0.38m^3 sand and 0.38m^3 gravel (1:3:3 mix)	Compressive cube strength, f_{cu} = 15-20MPa (low strength concrete)	
Reinforcement	9.4mm diameter iron bars spaced in a 10cm mesh	Steel profiled reinforcing bar, 12mm diameter	
Galvanised Steel Members	Galvanised A42 steel sections, 2mm thickness, Elastic Limit 260N/mm^2, Elastic Modulus 210kN/mm^2, density 78.5kN/m^3	Cold rolled 2mm thick galvanised steel, design yield strength 260N/mm^2, elastic modulus 210kN/mm^2, density 78.5kN/m^3	Assume members pre-drilled with holes to attach timber sub-frame.
Galvanised Steel Foundations	Galvanised steel sheet, 6mm thick + four anchor bolts	Assume 6mm thick steel, design yield strength 275N/mm^2, elastic modulus 210kN/mm^2, density 78.5kN/m^3	Assume anchor bolts are M20, 320mm long bolts of low strength steel – design yield strength 275N/mm^2
Plywood Flooring	18.75mm thick plywood, not treated	Plywood for floor to be 7/8" thick structural grade, 54/32 span rated, density 550kg/m^3	Plywood to be fixed using 8d nails spaced at 150mm centres
Timber Framing	Treated	Grade 2 Douglas Fir, density 530 kg/m^3, Young's Modulus 7584N/mm^2, bending strength 5.86N/mm^2	Member dimensions given are assumed to be as cut – no sacrificial allowance has been made
Galvanised Steel Roof Sheets	Aluminium/zinc galvanised corrugated sheeting, 28 6' gauge, nailed every three waves	Galvanised steel corrugated sheets, 75mm spacing x 18mm trough height. Sheet sizes 1.83m long by 8/10 corrugations wide (0.6-0.75m)	0.45mm sheet thickness assumed, weight 0.056kN/m^2
Plastic Sheeting	4 x 6m sheets	Polyethylene sheet with braided core (HDPE/LDPE) – 200g/m^2	Nailed to timber studs using 8d nails spaced at 150mm centres
Bolts	6.25mm diameter bolts, 65 or 100mm long	M6 bolts in S275 low grade steel	
Screws	Self-tapping screws	5mm diameter screws in low grade S275 steel	
Nails	75mm/62.5mm/37.5mm long	10d/8d/4d nails, 8d nails assumed	Assume 2 nails for all

		for cladding and flooring	timber connections

1.6.2 Cold Formed Galvanised Steel

The building frame has been fabricated using cold rolled, hot dip galvanised steel provided by Castelo Steel in Spain http://www.talleresestructurasmetalicas.com. Cold rolling is used to produce lightweight sections and work hardening and residual stresses from the process cause an increase in the yield strength at the expense of ductility and toughness.

The cold formed sections have been analysed in accordance with the method laid out in BS5950-5:1998. A yield strength for the steel of 260MPa and a tensile strength of 412MPa have been assumed (based on material properties given for the steel in Reference 7) and the yield strength enhanced by the appropriate factor to account for the increase in strength due to cold forming. The section capacities have been calculated assuming a thickness of 2mm.

1.7 Codes, Standards and References

General

The IBC (International Building Code) 2009 has been used as a basis for the design checks on the shelters since it is widely accepted worldwide, particularly for extreme loading cases such as earthquakes or strong winds. Other codes have however been referenced where appropriate or where the IBC was thought to be less applicable. This included the Eurocodes and local codes where appropriate.

Other references used:

- Standards referred to by IBC 2009 including: ASCE 7-10 (2010), NDS for Wood Construction, ACI 318 for Concrete, and AISC for Steel.
- UBC 1997 Volume 2 for preliminary wind calculations.
- BS5950-5: 1998 Structural use of steelwork in building – Code of practice for design of cold formed thin gauge sections.

1.8 Loads

1.8.1 Dead Loads

- Self-weight of structural materials applied in accordance with the densities specified in Section 1.6.1.

1.8.2 Live Loads

- For IBC compliancy live loads of 1.92kN/m^2 on the ground floor and 0.96kN/m^2 on the roof should be applied[1]. In this case however, no live load is assumed on the roof since there will be no maintenance access or snow/volcanic/sand load so it is not applicable. The live load allowance for the ground floor has been reduced to 0.9kN/m^2 since this represents a more realistic loading situation.

[1] 'International Building Code', ICC, 2009 – Table 1607.1.

1.8.3 Wind Loads

The wind loads have been calculated in accordance with the method specified in the IBC, referring to ASCE 7-10[1]. A basic wind speed of 60.4m/s for a 300 year return period[2] has been used. This speed exceeds that originally designed for but is considered a good baseline for transitional shelter design under the severe winds and frequent hurricanes in Haiti.

The internal, windward and leeward pressures have been evaluated directly from the square of the velocity using the factors detailed in the table below. The external and internal pressures have then been combined to give a number of design cases, the most critical of which have been checked for. An enclosed and a canopy case have been considered, for more details refer to Section 1.9.1.

Wind Directionality factor (K_d) from 26.6-1 for buildings	$K_d = 0.85$
Assume Surface Roughness C (open terrain with scattered buildings) from 26.7.2 to assign Exposure Category from 26.7.3 assuming shelter is not within 183m of the sea	Exposure Category C
Topographical Effects Factor (K_{zt}) from 26.8.2	$K_{zt} = 1.0$
Gust Effect Factor (G) assuming rigid building from 26.9.1	G = 0.85
Using the building height and Exposure Category along with factors from Table 26.9-1 in Table 27.3-1 to get the Velocity Pressure Exposure Coefficients (K_h and K_z)	$K_h = 0.85$ $K_z = 1.09$
Velocity Pressures (q_z and q_h) using Equation 27.3-1 $q_z = 0.613 K_z K_{zt} K_d V^2$	$q_z = 2.07$kPa $q_h = 1.61$kPa
Internal Pressure coefficients for a closed building, including gust factor (GC_{pi}) from Table 26.11-1 and External Pressure Coefficients (C_p) from Figure 27.4-1	Vary depending on wind direction and enclosed/canopy case

The Directional Procedure has been used to determine design wind pressures from Equation 27.4-1 using the factors detailed above. This gives a maximum uplift pressure for the canopy case of 2.4kPa and a maximum lateral force on the structure of 20.3kN in the transverse direction.

[1] See section 1.7 for further details.
[2] See Appendix A, Reference 8 for discussion on the choice of this return period which is thought to provide a good baseline for design. It may be considered too onerous for the design of lightweight transitional shelters and could be reduced at the discretion of the client. If a reduction is made then more robust hurricane proof structures must be provided elsewhere for shelter during loads of this magnitude.

1.8.4 Seismic Loads

The design response acceleration was determined using the PGA detailed in Section 1.2.2. This has been reduced according to the IBC[1] by 2/3 to give the design basis PGA of 0.67g, and a short period design acceleration determined based on the UBC methodology. The equivalent lateral force procedure has been used to calculate horizontal loads for design. The resulting base shear is only 7.2kN due to light weight of the materials used.

Design basis PGA (PGA_d) determined using IBC 2009 – Clause 11.4.6	$PGA_d = PGA*2/3$
Assume structure response in 0.5-1.5s period (UBC 16-3) to get S_{DS}	$S_{DS} = 2.5*PGA_d$
Assume Risk Category I (Table 1.5-1 low risk to human life in event of failure) in Table 11.6-1	Seismic Design Category D
Importance factor assuming risk category I – Table 1.5-2	$I_e = 1.0$
Assume no codified seismic lateral system – Table 12.2-1[2]	R = 1.0

1.9 Calculation Plan

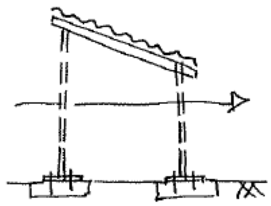
1.9.1 Design Methodology

The performance of each shelter has been assessed by checking that the structure as assumed from the information provided is safe for habitation. Relevant codes and standards have been used as the baseline for identifying appropriate performance/design criteria, but the structure has been checked against code requirements: if variations from this were made, assumptions and reasoning for lower factors of safety and alternative standards were justified. Logical reasoning was therefore used where necessary and upgrades suggested in order for the shelter to meet these criteria.

[1] Referencing ‘ASCE 7-10 – Minimum Design Loads for Buildings and Other Structures’, Chapters 11&12.
[2] Connections are not considered sufficient to resist lateral loads and no bracing has been provided.

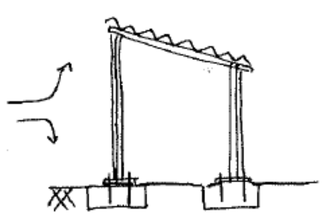
Two cases have been considered for the shelter:

i) CANOPY



- Assumes plastic sheeting tears or becomes unattached from structure
- wind uplift on roof only
- Laterally seismic loads will govern

ii) ENCLOSED



- Assumes plastic sheeting sufficiently fixed to transmit wind forces to members / plywood sheeting is used for walls.
- wind governs laterally.

Assumptions:

Two structural cases have been considered. The first scenario is one where the plastic sheeting ruptures or tears under wind loads and the structure therefore acts as a canopy with open sides. Seismic loads will act on the structure from its own self-weight but wind loads will govern. The second is an enclosed case where it is assumed that the plastic sheet wall covering has sufficient strength and is sufficiently fixed so that it will transfer high wind loads (i.e. a hurricane) to the steel frame without damage. This case is also applicable if the walls are upgraded with wood of sufficient strength.

1.9.2 Structural Checks

For a summary of the checks performed to assess the building, refer to Appendix C.

2 Results of Structural Assessment

2.1 General Key Findings

I. All members perform adequately under vertical and seismic loads only, but a lateral stability system must be provided in the form of properly nailed plywood or in plane bracing in the walls to resist both lateral seismic and wind loads.

II. The column foundations perform adequately in bearing under vertical, seismic and dead loads. The additional stub foundations supporting the floor fail in bearing under live loads if placed directly on the soil; a concrete pad must therefore also be provided under these foundations.

III. In the seismic case there is no overall uplift due to overturning or sliding of the foundations. Under wind loads the overall uplift on the columns cannot be resisted by the weight of the foundation alone and the overall lateral load exceeds the shear resistance of the foundation. An alternative foundation solution appropriate to the site soil type with more uplift capacity and shear resistance such as screw in ground anchors is therefore required (see Type 3, Annex 1).

IV. Under design wind pressures in the enclosed case it has been found that the columns and wall transoms fail in bending before plastic sheeting ruptures or tears/pulls out at nailed fixings. It has therefore been assumed that the enclosed case (see Case II, Section 1.9.1) governs for the shelter if the plastic sheeting is nailed to the timber studs using 8d nails at 150mm intervals.

V. Under horizontal design wind pressures the columns and wall transoms fail under the combined bending and axial loads, and as in the canopy case the roof beams and purlins are also overstressed under uplift forces. The roof sheeting and plastic sheeting can withstand these pressures if adequately fixed (see watch-its). More columns should be provided at a closer spacing to overcome these problems. The choice of wind speed as detailed in the assumptions is thought to explain these findings.

3 Conclusions and Recommendations

3.1 Assumptions

- Haiti experiences severe winds and a basic wind speed of 217 km/hr has been assumed along with Exposure Category C ('ASCE/SEI 7-10 – Minimum Design Loads for Buildings and Other Structures', ASCE, 2010). This is extremely high and it is difficult to resist these pressures in lightweight shelters.

  With more detailed knowledge of the site planning and placement of the shelters, the design wind pressures could be reduced by: intelligent grouping to reduce the Exposure Category to B (with the edge shelters designed for more stringent conditions) or assuming that lightweight structures will be damaged during a hurricane and providing a separate larger and heavier masonry hurricane shelter designed to withstand full hurricane loads.

- The maximum allowable floor live load is $0.9kN/m^2$ and it has been assumed that the roof of the structure will not be subjected to loading from volcanic ash, sand or snow.

- A stiff soil type (see Site Class D, '2009 International Building Code', ICC, February 2009) has been assumed in analysis of the structure. For sites where liquefaction may be a hazard (near river beds, coastal areas with sandy soils and high water tables), the shelters could be seriously damaged if soil liquefies in an earthquake but such damage is unlikely to pose a life safety risk to occupants due to the lightweight nature of the structure.
- If the plastic sheeting is nailed to the timber studs using 8d nails at 150mm intervals the columns and wall transoms will fail in bending before the plastic sheeting ruptures or tears/pulls out where it is nailed.
- During manufacture holes have been formed in steel members for the connection of the timber studs and sub-frame timber elements.
- Foundation base plates are 400*400*6mm thick (see Steel 1, I.1) and are held down to 800*800*400 plain concrete foundations by four M20 320mm long bolts (see I.1).
- It is assumed that all connections are of sufficient strength to transmit forces between members; the design and detailing of all connections is critical to the stability of the structure and should be checked for individual cases.

3.2 Conclusions

Performance Analysis	
Performance of frame under gravity loads alone is satisfactory. **However there is no lateral stability system and it is essential to provide in plane bracing in the roof and walls to make the structure safe (see C.2). Additional concrete foundations are required under stub column floor supports to take loads and prevent sagging.**	
Hazard	**Performance**
Earthquake – HIGH	Currently the shelter does not perform well under seismic loads. In-plane bracing is required in the walls and roof to provide lateral stability and prevent failure of the shelter in the event of an earthquake. However as the structure is lightweight, relatively flexible it attracts low seismic loads and overall will pose a low risk to the life safety of the occupants in the event of damage.
Wind – VERY HIGH	The shelter does not perform well under high wind loads. In addition to the provision of bracing, the foundation solution needs to be upgraded to prevent uplift and sliding. The column spacing must be decreased and the wall supports, roof purlins and roof beams strengthened to take uplift and lateral hurricane wind pressures.
Flood – HIGH	The shelter has a raised floor to prevent flood damage but no specific checks against standing water have been made.

Notes on Upgrades:

Possible upgrades to the shelter include the addition of plywood or corrugated metal sheeting walls which would be attached directly to the timber framing. In order to do this the following changes to the design would be needed to provide resistance to wind pressures:

- Provide in-plane bracing in roof and walls
- Provide concrete foundations under stub column floor supports
- Upgrade main foundations to prevent uplift and sliding
- Decrease column spacing and strengthen wall supports, roof beams and roof purlins

In order to upgrade the roof or walls with heavier and more substantial materials, member sizes should be increased accordingly and connections strengthened to take the increased gravity and seismic loads. Upgrading the shelter with masonry or other heavy materials is not recommended as they will attract high seismic loads causing the structure to perform poorly in an earthquake. Collapse of a heavy roof or unreinforced masonry walls poses a serious risk to the life safety of the occupants.

It should be noted when combining multiple shelter modules to create larger double pitched roof structures it is critical that the bracing is still included in the internal walls to ensure stability.

Watch-its for drawings: 'Change and Check'

A. CHANGE: Lateral stability can be improved by using ½" thick structural grade plywood with vertical framing spaced at 600mm and nailed 24/16 span rated 4-ply plywood with maximum 150mm nail spacing. Alternatively in plane diagonal bracing could be provided. In either case, header and primary floor beams must be strengthened (See C.2).

B. CHANGE: Provide in plane bracing in the roof and securely fasten roof sheet to the purlins using screws at every crest at eaves and ridge, and every other crest for rows in between (see C.3).

C. CHANGE: Add concrete pad footings underneath stub column floor supports to distribute bearing pressures on to the soil (Type 2, C.1).

D. CHANGE: Use alternative foundation solution to prevent uplift and sliding under wind loads (Type 4 or 5, C.1). In areas known to have higher local wind pressures design foundations and member sizes accordingly.

E. CHANGE: Decrease column spacing in accordance with design to recommended wind pressures, and increase number of foundations and rafters accordingly.

F. CHANGE: Strengthen roof purlins, roof beams and wall transoms to take hurricane wind pressures.

G. CHECK: Connect plastic wall sheeting to the timber studs using 8d nails at 150mm centres all round.

H. CHECK: Do not upgrade using masonry or cement blocks due to risk to life safety and increase in seismic force attracted to the structure.

I. CHECK: Design timber sub-frame to take wind pressures from walls back to steel members and therefore connect adequately to the steel frame.

J. CHECK: Provide hurricane straps at connections to secure wooden elements and steel elements against hurricane wind pressures.

K. CHECK: Check that the soil type for the shelter location is stiff, otherwise design foundations accordingly.

Appendix A – Source Information

1. 'Documentation for the initial seismic hazard maps for Haiti', A. Frankel, S. Harmsen, C. Mueller, E. Calais, J. Haase, USAID, April 2, 2010.

2. 'Wind Speed Maps for the Caribbean for Application with the Wind Load Provision of ASCE 7', P.J. Vickery & D. Wadhera, Applied Research Inc., Raleigh, NC, ARA Report 18108-1.

3. 'Haiti Earthquake, 12th January 2010', Zygmunt Lubkowski, Arup.

4. 'Transitional shelter Task Group Summary Information – Transitional shelter data sheet Haiti', CT & JA, December 2010.

5. 'Manual de montaje alojamiento temporal de emergencias', Cruz Roja Espanola & Talleres de estructura metalica, 2010.

6. 'CRUZ ROJA CASETAS NUEVAS – Alojamiento Progresivo', Anexo 1, Talleres de estructura metalica, 2010.

7. 'PO104024 – Alojamiento Progresivo NOTA TECNICA, Justificaion de calculus estructura', Talleres de estructura metalica, 2010.

8. 'Wind Loading for the design of transitional and permanent housing in Haiti', Damian Grant, Arup, 3rd June 2010.

Appendix B – Bill of Quantities

The table of quantities below is for the materials required to build the shelter. It does not take into account issues such as available timber lengths and allowances for spoilage in transport and delivery. Steel section thickness does not include galvanised coating.

Item (Dimensions in mm)	Material Spec.	No.	Total	Unit	Comments
Structure - Foundations					
Portland cement (42.5kg bags)	Concrete	3	3	bags	Quantity to be modified to reflect specification (see I.1)
Sand	Concrete	-	0.38	m^3	
Gravel (20mm aggregate)	Concrete	-	0.38	m^3	
Reinforcement bars 10 dia. (L=9.0m)	Rebar	4	4	bars	
Column base plate (300x300x6thk plate, 300 long 80x80x2thk column stub)	Steel 1	6	6	pieces	
Floor support base plate (100x100x6thk plate,435 long 40x40x2 column stub)	Steel 1	13	13	pieces	
Holding down bolts (20 dia. 320 long)	Bolts	24	24	pieces	
Main Structure					
Columns (80x80x2thk, L=3m)	Steel 3	3	9	m	
Columns (80x80x2thk, L=2.55m)	Steel 3	3	7.65	m	
Floor beams (40x40x2, L=2.995m)	Steel 3	4	11.98	m	
Roof cross beams (80x80x2, L=3.0m)	Steel 3	3	9	m	
Secondary Structure					
Floor joists (40x40x2, L=2.9m)	Steel 3	9	26.1	m	
Roof purlins (40x40x2, L=2.88m)	Steel 3	10	28.8	m	
Wall transoms (40x40x2, L=3.0m)	Steel 3	14	42	m	
Window framing (32.5x100, L=0.75m)	Timber 2	8	6	m	
Door framing (32.5x100, L=1.95m)	Timber 2	2	3.9	m	
Timber studs (32.5x100, L=3.35m)	Timber 2	45	151	m	To be modified for arrangement required
Plywood door (1.94m x 0.7m)	-	1	1	piece	
Covering – Wall, Roof and Floor					
Plywood flooring (21.8thk)	Plywood 2	-	18	m^2	
Steel sheeting (0.75m x 1.83m)	Sheet 1	40	54.9	m^2	
Plastic sheeting (6m x 4m)	Plastic	3	72	m^2	
Mosquito net	-	-	9	m^2	
Fixings					
Bolts, nuts + washers (20 dia. 320 long)	Bolts	12	12	pieces	
Bolts, nuts + washers (10 dia. 100 long)	Bolts	99	99	pieces	
Brackets (35wide, 70+20legs, 2thk)	Steel 3	70	70	pieces	
Bolts, nuts + washers (6.25 dia. 100 long)	Bolts	65	65	pieces	Location unclear
Steel angles (75x75x18.75)	-	150	150	pieces	To fix timber framing
Nails (10d)	Nails	1400	9.1	kg	Exact numbers will vary, minimum spacing given on drawings
Nails (8d)	Nails	1900	8.2	kg	
Nails (4d)	Nails	3800	5.4	kg	

Hinges	-	3	3	pieces	
Door latch + padlock	-	1	1	piece	
Self tapping screws	Screws	75	75	pieces	Exact numbers may vary, minimum spacing given on drawings
Tools Required					
Drill	-	1	1	piece	
Hammer	-	2	2	pieces	
Screw driver	-	2	2	pieces	
Tape measure	-	1	1	piece	
Spirit level	-	1	1	piece	
Plumb bob + 50m gut	-	1	1	piece	
Sockets (to fit 6.25/10/20 dia. bolts)	-	3	3	pieces	
Spanners (to fit 6.25/10/20 dia. bolts)	-	4	4	pieces	
Knitted Gloves	-	2	2	pieces	
Spade	-	1	1	piece	
Hand saw	-	1	1	piece	
Ladders	-	2	2	pieces	

Appendix C

Calculation Plan

1) Loading

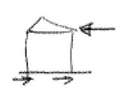
The steel members have been checked using strength design to BS5950-5 and relevant load tables. The loads described in Section 1.8 have been combined using the load factors given in the ASCE7-10, Section 2.3.2 since these apply to the wind pressures found using the ASCE method as described in Section 1.8.3.The critical timber elements and nail specifications have been checked using allowable stress methods in accordance with the NDS for wood construction. In this case the loads described in Section 1.8 have therefore been combined using the un-factored load cases described in the IBC (International Building Code) 2009, Section 1605.3.1.

2) Foundations

   a. Bearing pressure

   b. Uplift

   c. Base Shear



The effect of overturning must be included in the vertical force calculations.

3) Stability

   a. Overturning

   b. Transverse Stability – key members: columns, primary beams and bracing

   c. Longitudinal Stability – key members: columns, primary beams and bracing

4) Primary Members

Check members for a combination of vertical and lateral loads, including columns, beams and bracing. Check foundation base plates, foundations and holding down bolts for all load cases.

5) Secondary Members

Check members for a combination of vertical and lateral loads, including roof sheeting, purlins, flooring, joists, walling and wall supports.

6) Fixings – assumed to be strong enough to transmit member forces. Connections have been assumed to be pinned for analysis, including at column bases.