Shelter 7: Structural Assessment – Vietnam

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 7, built by the Vietnamese Red Cross in conjunction with local authorities. The information listed in Appendix A has been used in the assessment of this shelter.

Summary Information

Location: Vietnam, Ca Mau Province

Disaster: Various natural disasters from 1997 to the present day including typhoons and floods

Materials: Galvanised steel frame and zincalume corrugated roof sheeting

Material source: Concrete materials, blocks, plywood and roofing were sourced locally and the steel frame nationally

Time to build: 3 days

Anticipated lifespan: 5 years

Construction team: 6 people

Number built: 215

Approximate material cost per shelter: UNKNOWN

Approximate programme cost per shelter: 1500 CHF

Shelter Description

The shelter consists of a galvanised lightweight steel frame with plywood walls and a corrugated steel sheet roof. It has a covered area of 3.6 x 8.4m on plan and a living area of 3.6 x 7.2m, with a pitched roof of 16.5 degrees varying from 3.2m high at the eaves to 4.6m high at the ridge. There are two doors, one at the side and one at the front, and a cantilevered canopy projecting 1.3m beyond the door to form a porch. There are ten columns, six of which have screw in ground anchor foundations. Each column pair is connected by a braced truss to form a moment frame. The stability system of the shelter is formed by these three moment frames connected by two further moment frames on each edge of the building. There is also in plane steel tie bracing in the roof of the shelter underneath the roof sheeting. The base of the shelter is a 100mm thick concrete slab cast over the screw anchor foundations and floor tie beams. There is a low, 0.5m, brickwork wall providing a degree of flood protection, but is not connected to the frame and is not structural.

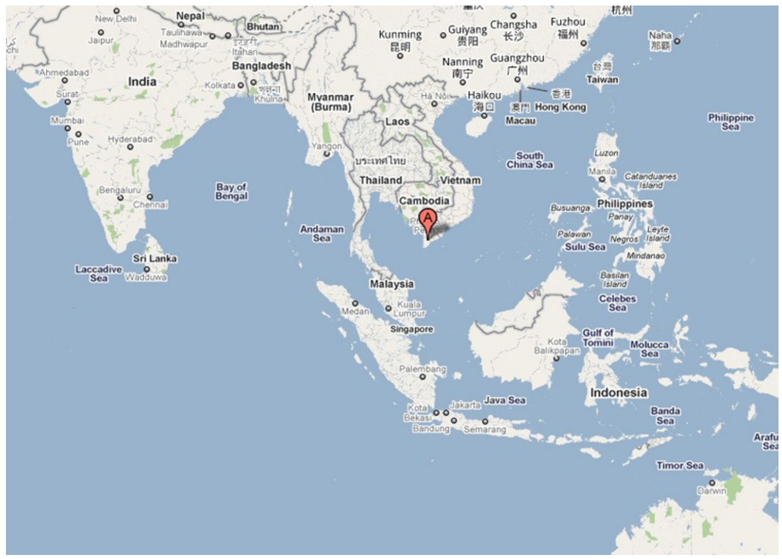
In many cases the shelters have been upgraded and extended using salvaged and recycled materials. The shelters were designed to be demountable and therefore reusable in alternative locations although the concrete floor and foundations are not reusable. As the cold formed steel sections are extremely thin, they are susceptible to corrosion, especially in salty environments, and the durability of the shelter is therefore poor.

1.2 Location and Geo-hazards

1.2.1 Location of Shelter

Vietnam, Ca Mau Province

Ca Mau Province of Vietnam (specifically Hogui village). We have assumed a coastal location in the Mekong Basin for all of the shelters, although the specific shelter location was in mangrove forest.



1.2.2 Hazards

A summary of the natural hazards faced in Ca Mau, Vietnam are given below[1]:

- LOW Earthquake. The Vietnamese Earthquake Design Code is based on the Eurocode 8[2] and gives a bedrock PGA value of 0.0256g for a 10% probability of exceedance in 95 years (equivalent to a 975 year return period[3]).
- MEDIUM Wind. Northern and central Vietnam are prone to typhoons resulting in high wind speeds that are not applicable to the southern Ca Mau province. The relevant region pressure value has therefore been used from the Vietnamese Wind Loading Code (see Reference 1, Appendix A) as described in Section 1.8.3.
- HIGH Flood. Monsoon and typhoon rainfalls coincide causing 1/5yr flooding in the Mekong Basin. Flash floods also occur.
- HIGH Landslide Risk. Previous history of landslides mud flows and erosion in the area.
- Hot humid climate prone to monsoons with low seasonal temperature variations.

1 See Appendix A, Reference 2.
2 'Eurocode 8: Design of Structures for Earthquake Resistance (EN 1998-1:2004).
3 This is has been reduced for a 475 year return period in line with the other shelters, see Section 1.8.4.

1.3 Geometry

The geometry was determined using the drawings and photographic information provided, see Figure 1.1 for key members and levels.

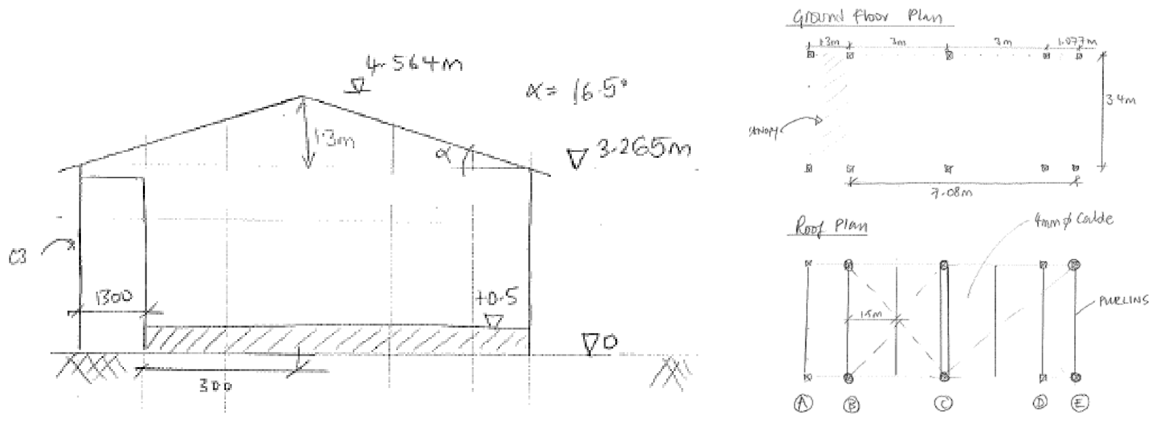


Figure 1.1 – Sketch of Geometry

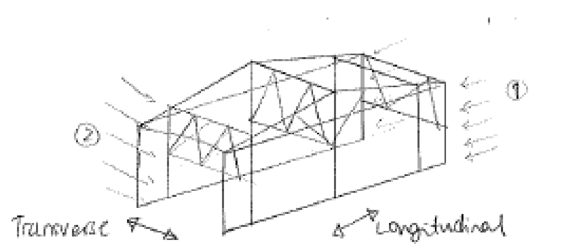


Figure 1.2 – 3D Sketch of Shelter

The shelter is 9m x 3.6m on plan and has a covered porch at the front and two doors, one at the front and one on the side. The structure rises to 4.6m high at the ridge from 3.2m at the eaves and has a double pitched roof with an angle of 16.5 degrees. It has a 0.5m high brick wall around three of the sides and has a window and ventilation strip on the front elevation.

There are ten columns longitudinally connected by the horizontal wall transoms that support the wall cladding. Six of these columns have screw anchor foundations and act in three pairs connected by trusses to form the transverse stability system. Longitudinally the two trussed frames on each side of the building provide the lateral stiffness. The other four columns are cast in to the base slab only. The roof consists of corrugated steel sheeting fixed to purlin members spanning between rafters that form the top chord of the longitudinal trusses. The roof contains in plane cable bracing to transmit lateral loads. The floor of the shelter consists of a 100mm thick concrete slab cast around the column bases upon which the brick wall sits. The walls have been assumed to be of plywood nailed to intermediate timber studs.

1.4 Structural System

- Vertical loads are carried by horizontal beam members back to vertical columns which transmit forces via screw foundations to the soil.
- Global stability in the transverse and longitudinal direction is provided by partially braced trussed frames that effectively act like moment frames. The lateral forces are transferred back to the foundations in shear by the screw anchors. Three frames in the transverse direction and two frames in the longitudinal direction have been considered (see Figure 1.2).

It should be noted that this is not a code compliant seismic or wind resistant lateral system, and that a low R value (see Section 1.8.4) has been chosen for seismic design since the thin cold formed steel sections have little ductility. Two 2D GSA models have been created to model the behaviour of the frames under lateral forces as shown in Figure 1.3.

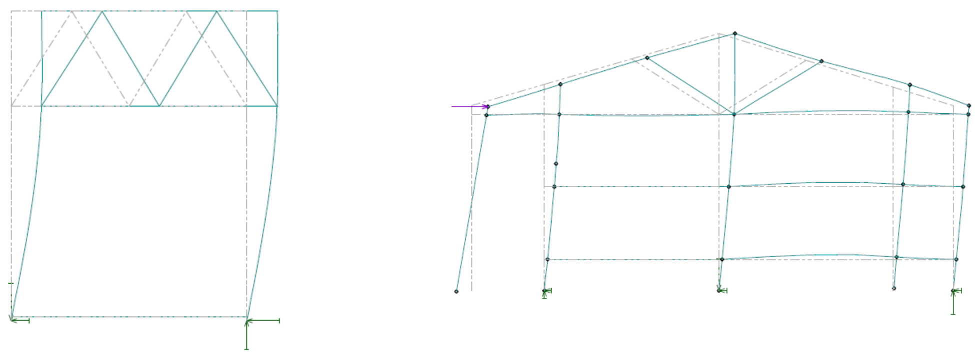
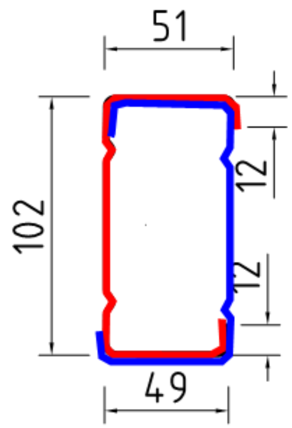


Figure 1.3 – Deflected Shape of Lateral System

Geometrical and Modelling Assumptions:

- We have reviewed the shelters built in Hogui only, designed for a non-flooding area in a forest. The sections used here are thinner than those used for the shelters in Kien Giang.
- All connections between members have been assumed to be pinned and fixed with two screws. Throughout it has been assumed that all connections are of sufficient strength to transmit axial forces between members.
- The column base plate is bolted to a plate welded to the top of the screw in ground anchor using 4 M12 bolts 30mm long in slotted holes. The column bases, screw foundation connections and cross beams have then been cast into a 100mm thick slab that forms the floor of the shelter and from which the 500mm high wall is supported. The column bases have been assumed to be pinned in analysis and the slab is assumed to be ground bearing with nominal steel reinforcement mesh.
- Screw anchor foundations consist of a 140mm diameter base plate and a cement filled 1200mm long, 60mm diameter, 3mm thick steel tube with a 400mm diameter, 3mm thick, 150 pitch helical screw plate.
- To model the longitudinal stability, the columns without screw anchor foundations have been assumed first to be pinned and then to be released and free to move in order to give an upper and lower bound on the other screw foundation forces. It is likely that in reality the feet of these columns will be provided with limited restraint from being cast into the thin concrete base.

- The walls of the shelter are formed from plywood sheets nailed to intermediate timber studs spaced at the appropriate intervals or an equivalent properly designed system that can transmit wind loads back to the frame without damage



- Columns formed of two channel sections have not been mechanically fixed and have been slid together so act independently of one another and do not form a composite section as shown in Figure 1.4.

Figure 1.4 Column 'Box' Section (0.75mm thick members)

1.5 Member Sizes

The table below shows the 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.

Primary Members		
C1/2/3/4	Main Columns (10)	102*51*1.5*0.75 Box from 2C sections
W5/6	Rafters	75*40*0.75 C
T2/2'/4/5	Lateral truss Brace	75*40*0.75 C
W4	Long. truss Brace	75*40*0.75 C
CB	Cable Bracing	4mm diameter rod
T3/W7	Bottom lat truss chord	75*40*0.75 C
W2/3	Bottom long. Truss chord	75*40*0.75 C
Secondary Members		
C5	Back wall columns	75*40*0.75 C
B1	Floor ties	102*51*0.75 C
B4	Door Framing	102*51*0.75 C
W1/2	Wall Transom	75*40*0.75 C
T1	Purlin	34*75 C with 12mm lip
P1	Purlin	1mm thk top hat

1.6 Materials

The materials were all sourced locally. The materials used in the shelter include concrete, galvanised cold formed steel members for the frame, corrugated roof sheeting, plywood for the walls, steel screw foundations and bricks or blocks for the low wall.

1.6.1 Material Assumptions

Type	IFRC Specification	Arup Assumption	Comments
Concrete	Concrete Slab	Compressive cube strength f_{cu} = 15-20MPa (low strength concrete)	Concrete mix as per specification (see I.1)
Steel Frame	Galvanised high tensile steel (frame) zincalume coated steel (wall supports and roof purlins) yield strength 550N/mm^2 for both	Cold formed galvanised steel, yield strength 550N/mm^2, density 78.5kN/m^3 and Young's modulus 205kN/mm^2	Cold formed steel has been assumed based on the thickness (0.75mm minimum) and nature of the sections specified – see Section 1.6.2 for further details
Steel Foundations	Hot dip galvanised steel	Galvanised hot rolled steel, yield strength 275N/mm^2, density 78.5kN/m^3	See Steel 1, I.1
Roof Sheeting	Zincalume corrugated sheeting with concealed fixings	Lightweight profiled steel sheet, 0.5mm thick, yield strength 275N/mm^2 (See Sheet 2, I.1) 2 x 4.3m sheets required	Nailed or screwed at every corrugation and every other corrugation in between through the crest of the corrugation
Plywood	Plywood walls	½" thick, 24/16 span rated, 4 ply, density 550kg/m^3 (Plywood 1, I.1)	Framing must be spaced at 600mm and a maximum nail spacing of 150mm used assuming 8d nails
Bricks	None	215x102x65 standard clay fired bricks assumed with maximum available mortar quality	Local standard to be used as appropriate
Timber	None	No. 2 Structural Grade treated Douglas Fir, density 530 kg/m^3, Young's Modulus 8274N/mm^2, bending strength 5.86N/mm^2	See Timber 2, I.1. Timber studs used where required to provide adequate support to plywood walls, suds to be screwed to steel members
Nails	None	Galvanised 8d twisted nails	Nails used for plywood walls
Screws	Self tapping screws – 10-24x22, 15-15x20, 12-14x20	Metric equivalent M5x0.08, M7 x 1.0 and M6 x1.0 screw assumed, minimum yield strength 275N/mm^2	Specified to fasten roof sheeting and frame elements
Bolts	M12x30	M12 steel bolts with minimum yield strength 275N/mm^2	

1.6.2 Cold Formed Steel Frame

It has been assumed that the thin channel sections used for the building frame are high strength cold formed galvanised steel with a yield strength of 550N/mm^2. The thin gauge of these sections (0.75mm thick) means that the design of the larger sections used for the columns is not covered in the recognised cold formed steel design codes. These standards have however been applied to the sections in order to approximate the section capacity. It should therefore be noted that the calculation of section capacities is approximate only to give an overview of the behaviour of the frame under the design load cases. The thickness of the galvanised coating is unknown.

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 also been referenced where appropriate or where the IBC is thought to be less applicable. This has included local codes where appropriate.

We have used a simplified static analysis assuming elastic behaviour to analyse the sections in order to overcome the problem of non-plastic brittle behaviour of cold formed sections compared to ordinary steel sections. It is therefore legitimate to use BS5950-5: 1998[1] to calculate the section properties.

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 some seismic design parameters.
- 'TCVN 2737:1995 Vietnamese Standards Loads and Actions – Design Code', 1995 – used since there is no accurate wind speed data of the correct return period for this region.
- BS5950-5:1998, Structural use of steelwork in building – Code of practice for design of cold formed thin gauge sections, BSI 2006.

[1] 'BS5950-5:1998, Structural use of steelwork in building – Code of practice for design of cold formed thin gauge sections, BSI 2006.

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. The total weight of the frame including the roofing is 850kg.

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 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. In this case the ground floor slab is ground bearing so this load does not need to be checked.

1.8.3 Wind Loads

The wind loads for Ca Mau have been found using the Vietnamese loading code as described in Sections 1.2.2 and 1.7. This divides Vietnam up into different wind pressure regions, here Zone IIA has been used to give the appropriate velocity pressure. The basic pressures are then factored as shown in the table below to give the design pressure.

W_o velocity pressure for Zone IIA (Annex D, Table 4) reduced for weak hurricane winds	W_o = 0.83kN/m^2
Assuming Exposure A (open terrain with no obstructions) and height less than 5m – Table 5	k = 1.07
Load factor	1.2
Aerodynamic Coefficients, c, for enclosed building	c – varies with wind direction and for each element

The resulting factored wind pressure on the windward face of the structure was found to be 0.85kPa. The un-factored maximum horizontal load on the structure is 35kN in the transverse direction. Local knowledge of higher wind speeds must be taken into account by using higher design pressures for specific shelter locations where necessary.

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

1.8.4 Seismic Loads

Seismic Loading has been considered in accordance with the IBC[1] using a short period design acceleration based on the UBC methodology. Stiff soil has been assumed (soil category D or Site Class D). The design response acceleration was determined using the PGA detailed in Section 1.2.2 modified for a 475 year return period.

Modify PGA for 475 year return period[2]	Z = 0.0185g
Assume Site Soil Category D[3] (20.3-1) and use PGA (Z) in UBC Table 16-Q interpolating	$C_a = 0.07g$
Assume structure response in 0.5-1.5s period (UBC 16-3) to get S_{DS}	$S_{DS} = 2.5C_a$
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 B
Importance factor assuming risk category I – Table 1.5-2	$I_e = 1.0$
Assume no codified seismic lateral system – Table 12.2-1[4]	R = 1.0

The equivalent lateral force procedure has been used to calculate horizontal loads for design. The resulting base shear is only 1.82kN due to light weight of the materials used. Therefore the wind loads are the dominant lateral load.

[1] 'ASCE 7-10 – Minimum Design Loads for Buildings and Other Structures', Chapters 11&12.
[2] 'Deriving the Seismic Action for Alternative return Periods according to the Eurocode 8', ZA Lubkowski, Arup London UK.
[3] In locations where liquefaction is a risk the Site Soil Category should be changed accordingly for seismic design.
[4] This assumes no reduction in seismic force that members are checked for, and therefore no plastic behaviour of the structure. This is consistent with the use of British Standards (see Section 1.6.2) to analyse section properties.

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 have been made, assumptions and reasoning for lower factors of safety and alternative standards have been justified. Logical reasoning has therefore been used where necessary and upgrades suggested in order for the shelter to meet these criteria.

Assumptions

For this shelter one structural form has been considered – the structure is enclosed and the roof and wall covering has sufficient strength to transmit wind loads to structural members without damage. The seismic loads acting on the structure are very small and the wind loads will govern.

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. The stability system of the shelter is not rational and unnecessarily complicated. For example not all of the trusses are aligned with the screw foundations and the lateral stiffness of these trusses is low. The deflection of the frame alone under wind and seismic loads is likely to be significant. The trusses are effectively redundant and the plywood walls provide a shear wall effect. It should therefore be checked that the shelters use properly nailed plywood walls (see Section 1.6.1) or that additional in-plane cross bracing is added in the walls.

II. The foundation solution and member sizes are adequate under all vertical loads.

III. Under seismic loads the capacity of the foundations is adequate. Under wind loads the bearing and uplift capacity of the screw foundations is sufficient but the shear capacity of the anchors is exceeded and stabilising shear plates should be included in the foundation solution.

IV. Under seismic loads most members perform adequately with the exception of the slender column sections in minor axis bending due to the longitudinal horizontal loads. The capacity of these sections could be improved by increasing the thickness or by connecting the interlocking channels to give a composite section. The thickness of other key members of the stability system including the bottom beam of the stability trusses in both directions should also be increased to give greater capacity.

V. Under wind loads the columns, rafters, wall and roof bracing, truss chords, purlins and wall transoms fail due to bending under wind pressures or exceedance of the very thin walled section compression capacity. The spacing of wall and roof supports should be decreased and all other sections strengthened, for example by increasing the thickness and/or using box sections where necessary.

3 Conclusions and Recommendations

3.1 Assumptions

- The bottoms of the columns that are not connected to the screw anchors are provided with limited restraint since they are cast in to the 100mm thick concrete floor slab. This slab is assumed to be reinforced with mesh only.
- The low brickwork wall is not to be connected to the structural steel frame so does not place any loads on it.
- All connections between members have been assumed to be pinned and fixed with two screws. Throughout it has been assumed that all connections are of sufficient strength to transmit forces between members.
- The columns are bolted to base plates and the screw in ground anchors have a stiffened 140mm diameter plate welded to the top of them. These plates are bolted together using 4, 30mm long, M12 bolts in slotted holes. The column bases, screw foundation connections and floor ties have then been cast into a 100mm thick slab that forms the floor of the shelter

and from which the 0.5m high wall is supported. The column bases have been assumed to be pinned in analysis and the slab is assumed to be resting on the ground.

- The screw in ground anchor foundations are as illustrated in C.1 (cement filled 1200mm long, 60mm diameter, 3mm thick steel tube with a 400mm diameter, 3mm thick, 150 pitch helical screw plate at the base).
- The walls of the shelter are formed from plywood sheets nailed to appropriately spaced intermediate timber studs screwed to the frame or an equivalent system that transmits wind loads back to the frame without being damaged.

3.2 Conclusions

Performance Analysis	
Performance under gravity loads is satisfactory. Bracing or plywood shear walls must be provided in order to increase the lateral stiffness of the structure.	
Hazard	**Performance**
Earthquake – LOW	The performance of the shelter under seismic loads is satisfactory. Damage is expected as the structure is flexible and has a low stiffness under lateral loads. Some steel members require strengthening to ensure the frame will not fail in the event of an earthquake. However, it is lightweight and attracts very low seismic loads so overall will pose a low risk to the life safety of the occupants. It is likely that the low brick wall will collapse in an earthquake.
Wind – MEDIUM	The performance of the shelter under wind loads is inadequate. The foundations require strengthening to improve the shear capacity. The spacing of the purlins and wall transoms should be decreased and the size and thickness of all members increased.
Flood – HIGH	Specific checks against standing water have not been made, however the provision of the 0.5m high brick wall helps to prevent flood damage.

Notes on Upgrades:

Upgrading the roof or walls with materials of a similar weight, for example lightweight metal sheets on the walls, would not change the structural performance of the shelter providing all cladding materials are adequately fixed to prevent damage under wind loads. In order to upgrade the roof or walls with heavier and more substantial materials, member sizes would need to be increased and the connections and foundation capacities checked under the increased gravity and seismic loads.

Upgrading the shelter with masonry or other very heavy materials above the current wall level 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.

Watch-its for drawings: 'Change or Check'

A. CHANGE: In-plane wall cross bracing or plywood shear walls required to improve the lateral stiffness of the shelter (see C.2).

B. CHANGE: Use foundation Type 5 (see C.1) as indicated but with stabiliser plates to increase the shear capacity.

C. CHANGE: Increase thickness of all steel members to standard section sizes in order to increase the durability of the structure.

D. CHANGE: Strengthen column members by using a box section or by connecting thicker interlocking channel sections to improve bending capacity under seismic and wind loads.

E. CHANGE: Do not offset roof bracing from columns. Bracing diameter should be increased to take forces from lateral wind loading.

F. CHANGE: Increase size of eaves beams on all sides to take seismic and wind loads.

G. CHANGE: Decrease spacing of wall transoms and change orientation so that bending under wind loads occurs around the stiff axis.

H. CHANGE: Decrease spacing of roof purlins according to design for critical uplift wind pressures.

I. CHANGE: Strengthen rafters, bracing and truss chords to take lateral wind loads, for example by using thicker sections.

J. CHANGE: Column heights can be reduced since no attic or mezzanine level is provided in the shelter. This would reduce the wall areas and therefore the wind loads on the shelter.

K. CHANGE: Reduce height and length of porch canopy to reduce the effect of uplift wind pressures on these members.

L. CHANGE: Canopy columns require screw in ground anchors (Type 5, C.1) as used for other columns in order to prevent uplift under the high local wind pressures.

M. CHECK: Roof sheeting should be 0.5mm thick and should span 3m with an intermediate purlin at the centre. Sheeting should be fixed using nails with washers at each crest at the eaves and ridge, and at every other corrugation crest on purlins in between.

N. CHECK: Plywood wall sheeting should be minimum ½" think, 24/16 span rating, 4 ply and must be nailed to intermediate timber studs spaced at a maximum of 600mm with 150mm nail spacing and 8d nails.

O. CHECK: Brick wall will not withstand seismic loads and may collapse in an earthquake event.

P. CHECK: The concrete mix for the base slab should be as per I.1 and a layer of mesh reinforcement used to prevent cracking and increase strength.

Q. CHECK: In areas known to have high local wind pressures adequate foundations and member sizes must be provided to account for this.

R. CHECK: A stiff soil type has been assumed in analysis of the structure. Rock or softer soils may not be suitable for screw in ground anchors so an alternative foundation solutions may be required.

S. CHECK: The design and detailing of all connections is critical to the stability of the structure and should be checked for individual cases.

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

Appendix A – Source Information

1. 'TCVN 2737:1995 Vietnamese Standards Loads and Actions – Design Code', 1995.
2. 'IFRC Hazard Assessment/Vietnam Ca Mau – Memorandum', 11th January 2011, Juliet Mian & Sasha Drozd (Arup).
3. 'Transitional Shelter Task Group – Summary Information Transitional Shelter Data Sheet', JA & CT, IFRC 2011.
4. 'Design Drawings Light Weight Steel Truss System Smartruss – Relocation housing project in Ca Mau Province', Drawings KC-01 to KC-07, Nuynh Nguyen Phung Co. Ltd., HCM City 05/2008.
5. 'Detail BOQ For 01 Unit – Swiss RC Low Cost House', Hogui – Ca Mau Province, HNP, 05/2008.
6. Photos and Images including 'HO RUI3-a', 'HO RUI 4-a' & 'Slide17 – Typhoon resistant Steel Housing in Vietnam', HNP, 2008.

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.

Item	Material Spec.	Quantity	Total	Unit	Comments
Structure - Foundations					
Screw Ground Anchors	See C.1	6	6	Pieces	See assumptions
Portland Cement	Concrete	15	15	Bags	42.5kg/bag, estimate only for 3.5m^3 concrete
Sand	Concrete	-	1.3	m^3	Estimate only
Gravel	Concrete	-	2.55	m^3	Estimate only
Wire Mesh Reinforcement	-	-	32.5	m^2	
Primary Structure					
Columns 2No. 100x50x0.75 Channels Interlocking to form box (L=3.3m)	Steel 4	4x2	26.4	m	
Columns 2No. 100x50x0.75 Channels Interlocking to form box (L=4.5m)	Steel 4	2x2	18.1	m	
Columns 2No. 100x50x0.75 Channels Interlocking to form box (L=3.7m)	Steel 4	4x2	29.2	m	
Rafters 75x40x0.75 Channel (L=4.5m)	Steel 4	4	18.0	m	
Transverse Beams 75x40x0.75 Channel (L=3.5m)	Steel 4	4	14	m	
Longitudinal Beams 75x40x0.75 Channel (L=4.3m)	Steel 4	2	8.6	m	
Longitudinal Beams 75x40x0.75 Channel (L=4.1m)	Steel 4	2	8.2	m	
Truss Bracing 75x40x0.75 Channel (L=1.65m)	Steel 4	8	13.2	m	
Truss Bracing 75x40x0.75 Channel (L=1.93m)	Steel 4	3	5.8	m	
Wall Bracing 75x40x0.75 Channel (L=1.7m)	Steel 4	4	6.8	m	
Secondary Structure					
Floor Ties 100x50x0.75 Channel (L=3.5m)	Steel 4	3	10.5	m	
Cable Bracing – 4mm dia. (L=5.3m or 4.5m) + Turn Buckle	Steel 4	4	19.6	m	
Wall columns 75x40x0.75 Channel (L=3.65m)	Steel 4	2	7.3	m	
Wall Transoms 75x40x0.75 Channel (L=3.0m)	Steel 4	6	18.2	m	
Wall Transoms 75x40x0.75 Channel (L=4.1m)	Steel 4	2	8.2	m	
Door Framing 75x40x0.75 Channel (L=2.3m)	Steel 4	2	4.6	m	
Purlins 75x35x0.75 Channel, 12mm lip (L=3.6m)	Steel 4	6	21.6	m	
Purlins 103x61x1.0 thk. (L=3.6m)	Steel 4	2	7.2	m	
Covering – Wall and Roof					
Plywood – 12.5mm thick	Plywood 1	-	90	m^2	
Roof Sheeting – 0.5mm thick (4.65x2m)	Sheet 2	4	34	m^2	
Ridge Capping 578x0.45mm thk. (L=3.8m)	Sheet 2	1	3.8	m	
Flashing 289 x 0.4mm thk (L=4.9m)	Sheet 2	4	19.6	m	
Timber studs	Timber 2	-	-	-	As required for walls

Fixings					
Bolts – M12x30	Bolts	35	35	Pieces	
Self tapping screws 10-24x22	Screws	200	200	Pieces	Roofing
Self tapping screws 15-15x20	Screws	80	80	Pieces	Flashing
Self tapping screws 12-14x20	Screws	500	500	Pieces	Frame
Cleat 100x50x1.9mm thick	Steel 1	4	4	Pieces	
Foundation Cleat 150x80x4mm thk	Steel 1	6	6	Pieces	
Nails – 8d	Nails	-	-	-	As required for walls
Fixing Strap	Steel 1		76	Pieces	For roof
Tools Required					
Drill	-	1	1	Pieces	
Hammer	-	2	2	Pieces	
Big Hammer	-	1	1	Pieces	
Screw Driver	-	3	3	Pieces	
Tape Measure, 5m	-	1	1	Pieces	
Spirit Level	-	1	1	Pieces	
Plumb bob + line	-	1	1	Pieces	
Spade	-	1	1	Pieces	
Hand saw	-	1	1	Pieces	
Ladders	-	2	2	Pieces	

Appendix C

Calculation Plan

1) Loading

   The steel members have been checked using strength design to BS5950-5. The loads described in Section 1.8 have therefore been combined using the load factors given in the IBC (International Building Code) 2009, Section 1605.2.1.

2) Stability

   a. Overturning due to lateral loads

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

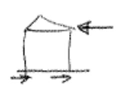
   c. Longitudinal Stability – key members: columns, beams and bracing

3) Foundations checked including consideration of the effects of overturning on the vertical forces:

   a. Bearing pressure

   b. Uplift

   c. Base Shear



4) Primary Members

Check members for all load cases including combinations of vertical and lateral loads. This includes:

- Columns
- Rafters and truss chords
- Bracing (roof, lateral and longitudinal)
- Ground floor slab for dead and live loads

5) Secondary Members

Check members for a combination of vertical and lateral loads, including purlins and wall transoms. Check capacity and fastening of plywood/corrugated sheet cladding and corrugated roof sheeting with current member configuration.

6) Fixings

It has been assumed that all connections between members are of sufficient strength to carry member forces. In analysis connections have been assumed to be pinned, including at column bases.