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Structural Design of Type V Construction For Antiterrorism Measures

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Policy Paper
Structural Design of Type V Construction
For Antiterrorism Measures

1. Scope. This paper states the structural design requirements for facilities built using type V construction as defined in the International Building Code (IBC) 2003 (specifically light-frame wood and cold-formed steel framing) to meet the provisions of UFC 4-010-01, DoD Minimum Antiterrorism Standards for Buildings, October 8, 2003, and UFC 4-023-03, Design of Buildings to Resist Progressive Collapse, January 28, 2005.

2. One and two story facilities with threat, standoff and level of protection conforming to UFC 4-010-01. Inhabited structures built in an area where no site specific threat has been identified, where the facility can be sited to provide the conventional construction standoff distances established in UFC 4-010-01 and where the owners can live with the very low or low levels of protection achieved by incorporation of the minimum standards, light-frame wood and cold-formed steel framing systems built in accordance with the International Building Code (IBC) are acceptable with the exception of windows, glazed doors and skylights.

   a. Glazing. Laminated glass windows, glazed doors and skylights must be designed in accordance with UFC 4-010-01, Standard 10.
   b. Connections. The connection of window frames, glazed door frames, and skylights to the surrounding structure is critical to prevent the glazed frame from separating from the structure and flying into the room. Designers will apply provisions of UFC 4-010-01, Standard 10; paragraph B-3.1.2.3, Connection Design.
   c. Supporting structural elements. Generally, the IBC provisions for framing around a window, door or skylight opening will not be adequate. The designer will design the structural support elements in accordance with UFC 4-010-01, Standard 10, paragraph B-3.1.2.4, Supporting Structural Elements.

3. One and two story facilities with threat, standoff distance or level of protection other than that indicated in UFC 4-010-01. Facilities with a site specific threat greater than those in the UFC, with standoff distances closer than the conventional construction standoff distances established in the UFC or with a level of protection higher than that stipulated in the UFC, will require a full blast analysis/design of the building envelope.

   a. The requirements listed in the IBC code are not necessarily sufficient to guarantee appropriate performance (level of protection) of the structure. All components of the exterior building envelope (walls, doors, windows and roofs) must be analyzed for their response to the blast loadings.
   b. All connections must be designed to support the full capacity of the members. For wood, this is typically the flexural shear and tension capacity of the member. For steel, shear (web crippling) and flexure (controlled by buckling) typically controls member capacity.

c. Windows and other glazed systems will need to be designed or tested to higher loads or better performance than required by the IBC or by standard 10 of the UFC.
d. Building lateral load resisting systems (diaphragms, shear walls, frames) need to be designed for blast loadings.

4. Facilities 3 or more stories tall. Paragraphs 2 or 3 above are still applicable to these facilities. In addition, these facilities must meet the provisions of UFC 4-023-03, Design of Buildings to Resist Progressive Collapse.
   a. Depending on level of protection for facility, use either tie forces or alternate path methods for analyzing capacity of members.
   b. Design load bearing walls to resist the vertical loads when the unsupported length is 2 times the floor to floor height. See Annex 2 of the attached commentary for more information.
   c. Analyze floor and roof structure for stress reversals due to upward blast loads.
   d. See Annex 2 of the attached commentary for more information on connection design.

5. Special inspection. Ensure special inspection procedures are specified for all structural elements that vary from the IBC minimum requirements and as required by UFC 4-023-03, Appendix G.

1. Scope. The Army wants to take advantage of the economies that can be achieved by allowing the use of wood and cold-formed steel framing systems in their permanent facilities. At the same time, they recognize the importance of incorporating the latest antiterrorism measures into the designs for these facilities. This paper intends to make it clear how the antiterrorism standards influence these designs and what extra measures must be taken above and beyond the requirements of the International Building Code (IBC) for wood and cold-formed steel structures.

2. One and two story facilities with threat, standoff and level of protection conforming to UFC 4-010-01. Blast pressures that a facility experiences vary with the explosive size and the standoff distance from the facility. Wood and light gage steel structures are marginally acceptable for the explosive sizes, the conventional construction standoff distances and the levels of protection established in UFC 4-010-01.
   a. Laminated glazing will be used on new facilities. Fragment retention film placed on annealed glass is reserved for existing facilities until the glazing can be replaced with laminated glazing.
   b. The connection of the frame to the structure is as important as the glazing to the frame.
   c. Assuming the frames are adequately fastened into the surrounding structural support elements, the designer must check the support elements to make sure they will carry these loads. This requires the use of the appropriate higher connection loads to be distributed to the supporting element based on tributary area of the glazed system if a static approach is used. If this results in the use of a structural component that is not feasible, then the designer may use a dynamic analysis. The dynamic analysis will apply the loads from the appropriate overall tributary area to the supporting element. The connection between the supporting element and the surrounding structure will then be designed to resist the ultimate capacity of the supporting element. Transfer of load into the rest of structure from the supporting element connection does not need to be accounted for in the structural analysis.

3. One and two story facilities with threat, standoff distance or level of protection other than that indicated in UFC 4-010-01. If the threat weapon increases, if the standoff distance is less than the conventional construction standoff distance, or the level of protection is higher than that specified in UFC 4-010-01, the standard IBC construction methods for these materials may not be acceptable. The designer must analyze the walls, windows, doors, roof and lateral load resisting system (diaphragms, shear walls, frames) for the effects of the design basis threat at the standoff distance available and to the level of protection desired. There are several tools available for the designer which makes the component analysis/design much easier. Those include SBEDS and CEDAW for the wall and roof components and HAZL for the windows. These are available for download from the PDC webpage at https://pdc.usace.army.mil/.

   Annex 2 shows a sampling of CEDAW runs for wall sections constructed out of both wood or cold formed steel studs and how they perform for a set of explosive weights and standoff distances. The table gives the standoff distances required to achieve the relative level of protection. You will notice that by adding extra mass to the wall section with something like brick veneer on the outside, the performance characteristics of these systems improve.

4. Facilities 3 or more stories tall. In addition to the provisions of paragraph 2 or 3 above, all the facilities 3 or more stories in height must conform to the requirements of UFC 4-023-03, Design of Buildings to Resist Progressive Collapse. This UFC requires meeting tie force requirements and, depending on the level of protection, an alternate path analysis to make sure that the impacts of a localized failure due to an explosive event does not propagate beyond the area affected by the initial damage or beyond certain specified limits.

   a. Stress reversal on the floors or roof can be caused by the explosive pressures entering the building and pushing up on the underside of the floor or roof. All floors and roofs must be analyzed for this possibility.

   b. In the event the floor is lost, the load bearing wall system must be analyzed as if its total length were 2 times the story height. This becomes a problem because of the connection details in platform framing methods. These wall studs are usually not continuous through the floor joint. The designer must alter the stud connection details to develop continuity of the wall system through the floor construction. One possible solution is the use of metal framing straps to generate some continuity between the wall above the floor and the wall below the floor.

   c. All other special connection details required to transfer loads from the tie forces or from the alternate path will need to be shown on the contract drawings.

   Annex 3 presents a series of CEDAW calculations for performance of wood studs subjected to axial loads.

5. Special inspections. When calculations indicate that construction methods, materials and/or connections are required that are different from those required by the IBC, the contract will need to include requirements for special inspection to verify that these special features are being installed according to the design. Special inspection should be in accordance with IBC Chapter 17, Section 1704 and UFC 4-023-03, Appendix G.

Annex 2
Stud Wall System Response from CEDAW Analysis

1. CEDAW analysis. A representative blast analysis was performed using the Component Explosive Damage Assessment Workbook (CEDAW) code to demonstrate the blast capabilities of wood and steel stud walls. Two wall systems were analyzed; one is a minimum mass wall similar to an exterior insulated foam system (EIFS), and a brick veneer façade on a stud wall. Wood and steel stud wall systems experiencing reflected and side-on blast loads were considered. The attached CEDAW analysis page indicates low mass systems in general experienced a blowout condition but when a brick veneer was added to the stud wall, a low or medium level of protection could be met.

Stud wall systems response from CEDAW Analysis

<u>All walls are 8-Foot high</u>	**Charge Location**			**Upper Bound values**				
	Conventional Construction			**Standoff Distance (ft) versus LOP***				
Wall System	**Charge**	**Standoff**		**Blowout**	**Low LOP**	**Med LOP**	**High LOP**	**LOP****
Wood studs								
2 X 4 @ 16	50	80	Reflected	142	178	253		**B**
2 X 6 @ 16	50	80	Reflected	147	171	228		**B**
2 X 8 @ 16	50	80	Reflected	126	147	184	315	**B**
2 X 4 @ 16 w/4" brick veneer	50	80	Reflected	31	45	82	171	**M**
2 X 6 @ 16 w/4" brick veneer	50	80	Reflected	60	75	102	199	**L**
2 X 8 @ 16 w/4" brick veneer	50	80	Reflected	63	76	98	181	**L**
2 X 10 @ 16 w/4" brick veneer	50	80	Reflected	60	70	89	158	**L**
2 X 12 @ 16 w/4" brick veneer	50	80	Reflected	55	64	80	136	**M**
2 X 4 @ 16 w/4" brick veneer	50	80	Side-On			18	87	**M**
2 X 4 @ 16	50	80	Side-On	70	91	136	296	**B**
2 X 6 @ 16	50	80	Side-On	76	94	125	236	**B**
2 X 6 @ 16 w/4" brick veneer	50	80	Side-On	13	22	47	105	**M**
2 X 8 @ 16	50	80	Side-On	66	78	99	181	**L**
2 X 6 @ 16	50	80	Reflected	147	171	228		**B**
2 X 6 @ 16	220	150	Reflected	376	439			**B**
2 X 6 @ 16	500	200	Reflected	602	692			**B**
2 X 6 @ 16	1000	250	Reflected	872				**B**
2 X 6 @ 16	50	80	Side-on	76	94	125	236	**B**
2 X 6 @ 16	220	150	Side-on	210	247	317	538	**B**
2 X 6 @ 16	500	200	Side-on	340	396	502		**B**
2 X 6 @ 16	1000	250	Side-on	505	581	713		**B**
Steel Studs								
6 X 2.5C 14ga w/4" brick veneer	50	80	Reflected	30	37	63	119	**M**
6 X 2.5C 14ga w/4" brick veneer	50	80	Side-on		14	28	62	**H**
6 X 2.5C 14ga w/4" brick veneer	220	150	Reflected	76	93	153	282	**M**
6 X 2.5C 14ga w/4" brick veneer	220	150	Side-on	32	44	80	154	**H**
6 X 2.5C 14ga w/4" brick veneer	500	200	Reflected	122	148	239	434	**L**
6 X 2.5C 14ga w/4" brick veneer	500	200	Side-on	59	75	129	240	**M**
6 X 2.5C 14ga w/4" brick veneer	1000	250	Reflected	178	216	342	605	**L**
6 X 2.5C 14ga w/4" brick veneer	1000	250	Side-on	92	115	186	340	**M**
6 X 2.5C 16ga w/4" brick veneer	50	80	Reflected	34	43	73	142	**M**
6 X 2.5C 16ga w/4" brick veneer	50	80	Side-on		16	33	76	**H**

*LOP= Level of Protection ** B=blowout; L=low level; M=medium level; H=high level

Annex 3

Stud Wall System Responses: Wood Column (ASD) Analysis

1. Column Analysis. A column stud wall analysis was performed for a range of 2-by lumber studs that are typically used it wood Type V construction to see what could be used to meet the minimum standards UFC. Column heights analyzed ranged from 8 to 12 feet for 2x4 through 2x12 sawn lumber. In order to account for the additional height resulting from the thickness of the floor joist system, the wood column lengths were increased by one foot. For the double length condition an 8-foot wall becomes 17-feet. Thus a load path is developed through the floor system to make the stud act as a single wooden column. It is assumed the design will take the axial through the vertical line of the stud and the moment would be taken by metal straps on each 2-inch face of the stud. Also, the slenderness ratio was calculated in accordance with AITC "Timber Construction Manual", 4th Edition. Thus L_e/d is less than or equal to 50, and the effective column length coefficient of $K_e = 1$ for a simple column was used. Therefore, a 2 X 6 stud wall is required to meet the double length criteria for single story wall heights up to 10-feet. A 2x 8 stud wall is required to meet this criteria for a single story up to 12-feet high when using balloon construction because of the column slenderness requirement (see the attached Excel Spreadsheet printout).

2. Connections. The designer must detail a connection to deliver the full shear capacity of the member when loaded to its full moment capacity.

   When maximum shear is equal to:

   $$V = w\ L / 2$$

   Then the maximum shear stress is:

   $$f_v = 3\ V / (2\ bd)$$

   The maximum load per foot based on the horizontal shear capacity is:

   $$w = 4\ bd\ f_v / (3\ L)$$

   V = shear load on the end of a simply supported, uniformly loaded member, pounds
   w = the maximum uniform load on the member, pounds / foot
   L = the span, feet
   b = member width, inches
   d = member depth, inches
   f_v = horizontal shear stress, psi

Wood Column Buckling

Buckling formulas are from AITC, "Timber Construction Manual", 4th Edtion. This is in agreement with the UFGS 06100 Rough Carpentry.

Loads (psf)		Load coef	Design load
10	D = dead	1.2	12
	L = live		
40	floor	0.5	20
20	roof	0.5	10
20	W = wind	0.2	4

Column description (first floor wall)

16	s = joist spacing (inches)
16	l = joist span (feet)
576	Pa = Axial load per floor on column (lbs)
1	number of floors
1	roof

Material properties

Compression

780	σ = allowable stress (UFGS)
1,014	CD*σ
1,200,000	E = modulus of elasticity (UFGS)

1.3	CD	for 1-day duration
0.3	KCE	for sawn lumber
0.8	c	for sawn lumber
1.0	Ke	Effective column length

Bending

1200	σ = allowable stress (UFGS)
1,560	CD*σ
1,200,000	E = modulus of elasticity (UFGS)

All material is 2 X d

L = length d = least dim, col

AITC "5-15"

Mbw = bending moment from wind (lbs-in)

ft	inches	d	(L/d)	FCE	FCE/Fc	Cp	fca	fc	sec mod	fbw / Mbw	fba	fc/fca	fbw/fba	sum	L/d<50
										512					
8	96	4	27.4	479	0.4719	0.4136	110	419	3.06	167	1,560	0.26	0.11	0.37	OK
		6	17.5	1182	1.1653	0.7411	70	751	7.56	68	1,560	0.09	0.04	0.14	OK
		8	13.2	2053	2.0249	0.8692	51	881	13.14	39	1,560	0.06	0.02	0.08	OK
		10	10.4	3342	3.2961	0.9274	40	940	21.39	24	1,560	0.04	0.02	0.06	OK
		12	8.5	4944	4.8756	0.9536	33	967	31.64	16	1,560	0.03	0.01	0.04	OK
										2312					
17	204	4	58.3	106	0.1045	0.1022	110	104	3.06	755	1,560	1.06	0.48	NG	NG
		6	37.1	262	0.2581	0.2425	70	246	7.56	306	1,560	0.28	0.20	0.48	OK
		8	27.2	487	0.4799	0.4193	51	425	13.14	176	1,560	0.12	0.11	0.23	OK
		10	21.5	781	0.7699	0.5951	40	603	21.39	108	1,560	0.07	0.07	0.14	OK
		12	17.7	1144	1.1282	0.7310	33	741	31.64	73	1,560	0.05	0.05	0.09	OK
										648					

9	108	4	30.9	378	0.3729	0.3383	110	343	3.06	212	1,560	0.32	0.14	0.46	OK
		6	19.6	934	0.9208	0.6618	70	671	7.56	86	1,560	0.10	0.05	0.16	OK
		8	14.4	1736	1.7121	0.8388	51	851	13.14	49	1,560	0.06	0.03	0.09	OK
		10	11.4	2785	2.7470	0.9099	40	923	21.39	30	1,560	0.04	0.02	0.06	OK
		12	9.4	4082	4.0254	0.9424	33	956	31.64	20	1,560	0.03	0.01	0.05	OK
										2888					
19	228	4	65.1	85	0.0837	0.0822	110	83	3.06	943	1,560	1.32	0.60	NG	NG
		6	41.5	209	0.2066	0.1969	70	200	7.56	382	1,560	0.35	0.24	0.59	OK
		8	30.4	390	0.3842	0.3472	51	352	13.14	220	1,560	0.15	0.14	0.29	OK
		10	24.0	625	0.6164	0.5101	40	517	21.39	135	1,560	0.08	0.09	0.16	OK
		12	19.8	916	0.9032	0.6548	33	664	31.64	91	1,560	0.05	0.06	0.11	OK

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AITC "5-15"

Mbw = bending moment from wind (lbs-in)

ft	inches	d	(L/d)	FCE	FCE/Fc	Cp	fca	fc	sec mod	fbw / Mbw	fba	fc/fca	fbw/fba	sum	L/d<50
										800					
10	120	4	34.3	306	0.3020	0.2802	110	284	3.06	261	1,560	0.39	0.17	0.55	OK
		6	21.8	756	0.7458	0.5829	70	591	7.56	106	1,560	0.12	0.07	0.19	OK
		8	16.6	1314	1.2959	0.7722	51	783	13.14	61	1,560	0.07	0.04	0.10	OK
		10	13.0	2139	2.1095	0.8757	40	888	21.39	37	1,560	0.05	0.02	0.07	OK
		12	10.7	3164	3.1204	0.9226	33	935	31.64	25	1,560	0.04	0.02	0.05	OK
										3528					
21	252	4	72.0	69	0.0685	0.0675	110	68	3.06	1152	1,560	1.60	0.74	NG	NG
		6	45.8	171	0.1691	0.1628	70	165	7.56	467	1,560	0.42	0.30	0.72	OK
		8	33.6	319	0.3145	0.2907	51	295	13.14	268	1,560	0.17	0.17	0.35	OK
		10	26.5	512	0.5046	0.4368	40	443	21.39	165	1,560	0.09	0.11	0.20	OK
		12	21.9	750	0.7394	0.5796	33	588	31.64	112	1,560	0.06	0.07	0.13	OK
										1152					

12	144	4	41.1	213	0.2097	0.1998	110	203	3.06	376	1,560	0.54	0.24	0.78	OK
		6	26.2	525	0.5179	0.4461	70	452	7.56	152	1,560	0.15	0.10	0.25	OK
		8	19.2	977	0.9631	0.6778	51	687	13.14	88	1,560	0.07	0.06	0.13	OK
		10	15.2	1567	1.5452	0.8168	40	828	21.39	54	1,560	0.05	0.03	0.08	OK
		12	12.5	2296	2.2643	0.8861	33	898	31.64	36	1,560	0.04	0.02	0.06	OK
										5000					
25	300	4	85.7	49	0.0483	0.0478	110	49	3.06	1633	1,560	2.26	1.05	NG	NG
		6	54.5	121	0.1193	0.1163	70	118	7.56	661	1,560	0.59	0.42	1.02	NG
		8	40.0	225	0.2219	0.2107	51	214	13.14	380	1,560	0.24	0.24	0.48	OK
		10	31.6	361	0.3560	0.3248	40	329	21.39	234	1,560	0.12	0.15	0.27	OK
		12	26.1	529	0.5217	0.4487	33	455	31.64	158	1,560	0.07	0.10	0.17	OK