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LL13 7YL

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Website: www.opusinternational.co.uk

Project No: V-R6306.0

Structural Engineers Calculations

For

Proposed New Dwelling

At

The Lodge, Croes Howell

Refer to the Architect's drawings for layout & dimensions.

Do not take dimensions from these calculations.

Design Standards – Current versions at time of design

BS EN 1990	Eurocode - Basis of Structural Design
BS EN 1991	Eurocode 1: Actions on Structures
BS EN 1992	Eurocode 2: Design of Concrete Structures
BS EN 1995	Eurocode 5: Design of Timber Structures
BS EN 1997	Eurocode 7: Geotechnical Design

All Relevant National Annexes

Issue	Date	Documents	Issued	Reviewed
0	01/07/2016	First Issue	IW	LM

Calculation Sheet

Project/Reference No: UR6306

Sheet No: 01

Project and Description: THE LODGE CROES HOWELL
NEW DWELLING

Office: WREXHAM

Designed: IW

Checked:

PROVIDE STRUCTURAL CALCULATIONS FOR PROPOSED NEW DWELLING AT THE LODGE, CROES HOWELL, WREXHAM

PROPOSED 4 BEDROOM DWELLING IS TO REPLACE EXISTING BUILDING ON PLOT, ALONG WITH NEW 2 BAY GARAGE

CALCULATIONS REQUIRED FOR THE FOLLOWING ELEMENTS

- SUBSTRUCTURE DESIGN - FOUNDATIONS + FLOOR SLABS
- STEEL BEAMS TO KITCHEN
- TRIMMING TO STAIRS + ASSOCIATED LINTELS BELOW
- ISOLATED PIER TO GARAGE DOOR

DESIGN TO UNDERTAKEN TO ALL RELEVANT EUROCODES ✓

NO GROUND INVESTIGATION INFORMATION FOR THE SITE AVAILABLE HOWEVER FROM AVAILABLE BOREHOLE LOGS FROM NEARBY INDICATE A BED OF GRAVEL APPROX 20m THICK OVERLAYING CLAY ✓

∴ ASSUMED BEARING PRESSURE CAPACITY = 100 kN/m^2 ✓

AS CLAY IS SUFFICIENTLY DEEP NO IMPACT ON THE DESIGN FROM EXISTING TREES ON THE SITE

PROPERTY SITS WITHIN RADON ZONE REQUIRING FULL RADON PROTECTION

To be read in conjunction with
Arboricultural Report

Site to be left clear of debris on completion and all finishes made good to client approval.



	STATUS	LEVEL OF MODIFICATION (LMD)	DATE
P4	Found as building control comments	REVISION	04/11/02
P3	Found as planning comments	REVISION	23/04/02
P2	Found as TIA Report	REVISION	04/02/02

REVISION / DESCRIPTION (if T/O, LMD)

ROAD PLAN OF WORKING WORKSHEET

PURPOSE OF ISSUE - AT FOR -

CARDIFF
029 2076 2511

LONDON
0207 138 3560

WREXHAM
01978 357 887

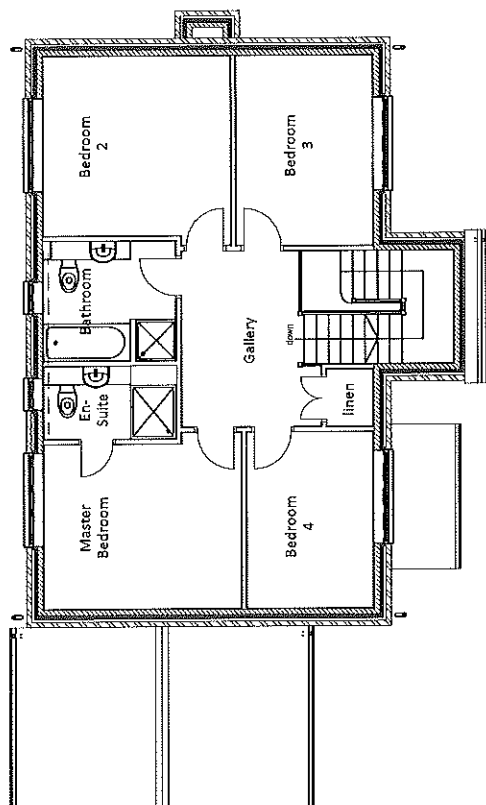
CLIENT
Malka Private Electrical

PROJECT TITLE: Parbecment Nesting The | rds Crestwell | are Bnbn

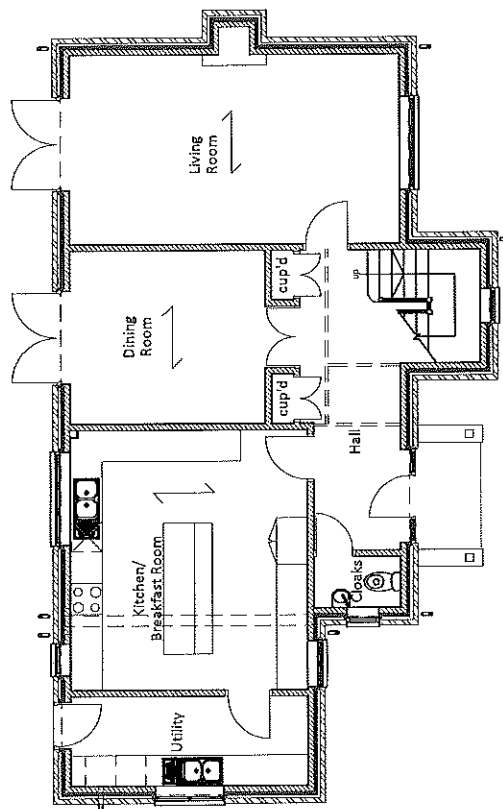
DRAWING TITLE
Dressed C&G Elev

DATE	SCALE #10	DRAWN	CHECKED	APPROVED
02.15	120		COH	
PROJECT NO. DRAWING NO. REVISION				
PROJECT - COLUMBIA - 2001 - 2002 - 2003 - 2004 - 2005 - 2006 - 2007 - 2008 - 2009 - 2010 - 2011 - 2012 - 2013 - 2014 - 2015 - 2016 - 2017 - 2018 - 2019 - 2020 - 2021 - 2022 - 2023 - 2024 - 2025 - 2026 - 2027 - 2028 - 2029 - 2030 - 2031 - 2032 - 2033 - 2034 - 2035 - 2036 - 2037 - 2038 - 2039 - 2040 - 2041 - 2042 - 2043 - 2044 - 2045 - 2046 - 2047 - 2048 - 2049 - 2050 - 2051 - 2052 - 2053 - 2054 - 2055 - 2056 - 2057 - 2058 - 2059 - 2060 - 2061 - 2062 - 2063 - 2064 - 2065 - 2066 - 2067 - 2068 - 2069 - 2070 - 2071 - 2072 - 2073 - 2074 - 2075 - 2076 - 2077 - 2078 - 2079 - 2080 - 2081 - 2082 - 2083 - 2084 - 2085 - 2086 - 2087 - 2088 - 2089 - 2090 - 2091 - 2092 - 2093 - 2094 - 2095 - 2096 - 2097 - 2098 - 2099 - 2100 - 2101 - 2102 - 2103 - 2104 - 2105 - 2106 - 2107 - 2108 - 2109 - 2110 - 2111 - 2112 - 2113 - 2114 - 2115 - 2116 - 2117 - 2118 - 2119 - 2120 - 2121 - 2122 - 2123 - 2124 - 2125 - 2126 - 2127 - 2128 - 2129 - 2130 - 2131 - 2132 - 2133 - 2134 - 2135 - 2136 - 2137 - 2138 - 2139 - 2140 - 2141 - 2142 - 2143 - 2144 - 2145 - 2146 - 2147 - 2148 - 2149 - 2150 - 2151 - 2152 - 2153 - 2154 - 2155 - 2156 - 2157 - 2158 - 2159 - 2160 - 2161 - 2162 - 2163 - 2164 - 2165 - 2166 - 2167 - 2168 - 2169 - 2170 - 2171 - 2172 - 2173 - 2174 - 2175 - 2176 - 2177 - 2178 - 2179 - 2180 - 2181 - 2182 - 2183 - 2184 - 2185 - 2186 - 2187 - 2188 - 2189 - 2190 - 2191 - 2192 - 2193 - 2194 - 2195 - 2196 - 2197 - 2198 - 2199 - 2200 - 2201 - 2202 - 2203 - 2204 - 2205 - 2206 - 2207 - 2208 - 2209 - 2210 - 2211 - 2212 - 2213 - 2214 - 2215 - 2216 - 2217 - 2218 - 2219 - 2220 - 2221 - 2222 - 2223 - 2224 - 2225 - 2226 - 2227 - 2228 - 2229 - 2230 - 2231 - 2232 - 2233 - 2234 - 2235 - 2236 - 2237 - 2238 - 2239 - 2240 - 2241 - 2242 - 2243 - 2244 - 2245 - 2246 - 2247 - 2248 - 2249 - 2250 - 2251 - 2252 - 2253 - 2254 - 2255 - 2256 - 2257 - 2258 - 2259 - 2260 - 2261 - 2262 - 2263 - 2264 - 2265 - 2266 - 2267 - 2268 - 2269 - 2270 - 2271 - 2272 - 2273 - 2274 - 2275 - 2276 - 2277 - 2278 - 2279 - 2280 - 2281 - 2282 - 2283 - 2284 - 2285 - 2286 - 2287 - 2288 - 2289 - 2290 - 2291 - 2292 - 2293 - 2294 - 2295 - 2296 - 2297 - 2298 - 2299 - 2300 - 2301 - 2302 - 2303 - 2304 - 2305 - 2306 - 2307 - 2308 - 2309 - 2310 - 2311 - 2312 - 2313 - 2314 - 2315 - 2316 - 2317 - 2318 - 2319 - 2320 - 2321 - 2322 - 2323 - 2324 - 2325 - 2326 - 2327 - 2328 - 2329 - 2330 - 2331 - 2332 - 2333 - 2334 - 2335 - 2336 - 2337 - 2338 - 2339 - 2340 - 2341 - 2342 - 2343 - 2344 - 2345 - 2346 - 2347 - 2348 - 2349 - 2350 - 2351 - 2352 - 2353 - 2354 - 2355 - 2356 - 2357 - 2358 - 2359 - 2360 - 2361 - 2362 - 2363 - 2364 - 2365 - 2366 - 2367 - 2368 - 2369 - 2370 - 2371 - 2372 - 2373 - 2374 - 2375 - 2376 - 2377 - 2378 - 2379 - 2380 - 2381 - 2382 - 2383 - 2384 - 2385 - 2386 - 2387 - 2388 - 2389 - 2390 - 2391 - 2392 - 2393 - 2394 - 2395 - 2396 - 2397 - 2398 - 2399 - 2400 - 2401 - 2402 - 2403 - 2404 - 2405 - 2406 - 2407 - 2408 - 2409 - 2410 - 2411 - 2412 - 2413 - 2414 - 2415 - 2416 - 2417 - 2418 - 2419 - 2420 - 2421 - 2422 - 2423 - 2424 - 2425 - 2426 - 2427 - 2428 - 2429 - 2430 - 2431 - 2432 - 2433 - 2434 - 2435 - 2436 - 2437 - 2438 - 2439 - 2440 - 2441 - 2442 - 2443 - 2444 - 2445 - 2446 - 2447 - 2448 - 2449 - 2450 - 2451 - 2452 - 2453 - 2454 - 2455 - 2456 - 2457 - 2458 - 2459 - 2460 - 2461 - 2462 - 2463 - 2464 - 2465 - 2466 - 2467 - 2468 - 2469 - 2470 - 2471 - 2472 - 2473 - 2474 - 2475 - 2476 - 2477 - 2478 - 2479 - 2480 - 2481 - 2482 - 2483 - 2484 - 2485 - 2486 - 2487 - 2488 - 2489 - 2490 - 2491 - 2492 - 2493 - 2494 - 2495 - 2496 - 2497 - 2498 - 2499 - 2500 - 2501 - 2502 - 2503 - 2504 - 2505 - 2506 - 2507 - 2508 - 2509 - 2510 - 2511 - 2512 - 2513 - 2514 - 2515 - 2516 - 2517 - 2518 - 2519 - 2520 - 2521 - 2522 - 2523 - 2524 - 2525 - 2526 - 2527 - 2528 - 2529 - 2530 - 2531 - 2532 - 2533 - 2534 - 2535 - 2536 - 2537 - 2538 - 2539 - 2540 - 2541 - 2542 - 2543 - 2544 - 2545 - 2546 - 2547 - 2548 - 2549 - 2550 - 2551 - 2552 - 2553 - 2554 - 2555 - 2556 - 2557 - 2558 - 2559 - 2560 - 2561 - 2562 - 2563 - 2564 - 2565 - 2566 - 2567 - 2568 - 2569 - 2570 - 2571 - 2572 - 2573 - 2574 - 2575 - 2576 - 2577 - 2578 - 2579 - 2580 - 2581 - 2582 - 2583 - 2584 - 2585 - 2586 - 2587 - 2588 - 2589 - 2590 - 2591 - 2592 - 2593 - 2594 - 2595 - 2596 - 2597 - 2598 - 2599 - 2600 - 2601 - 2602 - 2603 - 2604 - 2605 - 2606 - 2607 - 2608 - 2609 - 2610 - 2611 - 2612 - 2613 - 2614 - 2615 - 2616 - 2617 - 2618 - 2619 - 2620 - 2621 - 2622 - 2623 - 2624 - 2625 - 2626 - 2627 - 2628 - 2629 - 2630 - 2631 - 2632 - 2633 - 2634 - 2635 - 2636 - 2637 - 2638 - 2639 - 2640 - 2641 - 2642 - 2643 - 2644 - 2645 - 2646 - 2647 - 2648 - 2649 - 2650 - 2651 - 2652 - 2653 - 2654 - 2655 - 2656 - 2657 - 2658 - 2659 - 2				

Design that increasingly was needed. Throughout the design stage of this project, in accordance with the company procedures and technical personnel, Wilson necessarily provided all items of this type applicable to the design. Items were borrowed and then returned, although it would be better, if the design team had a good idea of which a component design or technology applied to the same or not included. The drawing is to be used in conjunction with the Pre-Construction Information Pack and related documents prepared in accordance with the British Construction Pack and the Government Pack 2007, and all technical construction details as provided.



First Floor Plan



Ground Floor Plan

PR	Need as Prioror comments	REV	STATUS
REV	DESCRIPTION (BY) / CHG (AP)	DATE	
	ROSA PLAN OF WORK-2015 WORKSTAGE	LEVEL OF MODEL DEFINITION (LMD)	
	PURPOSE OF ISSUE- FIT FOR...	STATUS	

11. **Bar of London (Incorporated)** - 1881. The first incorporation.

GARDIFF
029 2070 2511
www.lawtey.co.uk

LONDON
0207 138 3500

WREXHAM
01978 257 887

CLIENT
Miles Pyro Electrical

PROJECT TITLE
Displacement of

PROJECT TITLE
Replacement Dwelling, The Lodge, Crosswell Lane, Burton

DRAWING TITLE
Floor Plans

DRAWING TITLE
Floor Plans

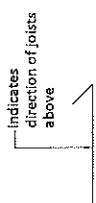
DATE	SCALE	DOWN	CHECKED	APPROVED
02.15	1:00	COH	-	
PROJECT NO	PROJECT DESCRIPTION	DATE	TIME	REV
N1081	CHHL-LAW-X-DFP-A-R-061101			P2
DO NOT SCALE! Any discrepancy or error concerning this drawing should be referred to the architect.				
Copyright © LUMLEY LIMITED This drawing is the property of LUMLEY LIMITED and is not to be used for any other purpose without the written consent of LUMLEY LIMITED.				

Note all dimensions to face of stud/structure.

Kitchen

Utility Sink; 40m diam combined sink/washing machine waste with 75mm deep seal anti syphon trap to 50mm diam vertical pipe connecting to below ground drainage via polypropylene adaptor at floor level.

Cloaks
WWHB; 32mm diam waste with 75mm deep seal anti syphon trap to stub stack.
WC; 100mm diam connection to stub stack to below ground drainage, terminating above flood level.



P2	Read as Primary comments	RTN	DATE
	REV DESCRIPTION (RTN) OR (APP ROSA PLAN OF WORK 2013 WORKSTAGE LEVEL OF MODEL DEFINITION (LOD)		
	PURPOSE OF ISSUE - RTN FOR ...		STATUS

1. *Agave* (100%)

CARDOFF 029 2078 2515
LONDON 0207 138 3560
WREXHAM 01978 357 887

www.lawrey.co.uk

CLIENT
Miles Pryde Electrical

FULLY TRAINED

PROJECT TITLE
Replacement Dwelling, The Lodge, Croxted Lane, Burton

[illegible]

GROUND FLOOR PLAN

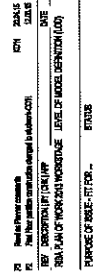
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02.15	1:50	CON.	-	-
PROJECT NO		DRAWINGS NO		
		PLOT - COMMERCIAL - ZONE - 100% - 100% - 100%		
N1681	GHH-LAWI-X-500-D-FP-AR-061102	P2	REV	

DO NOT SCALE Any dimension is given according to drawing sheets as indicated in the schedule

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During the measurements we carried out, described the design steps of the project, in accordance with the company procedures and technical standards. We were increasingly conscious of some of the key factors in the design (how flexible and how adaptable, simplified or suited to others). We had gained data on a completed design's customer trial in the previous year, and we had been involved. The design is to be an aid in cooperation with the PC-connection between the PC and the related documents required for acceptance with the external transmission design and the new and Regulators' data and is applicable to various industries in a similar way.

En-Suite	Bathroom
Shower: 40mm diam waste with 75mm deep seal and syphon trap above floor to stub stack in corner.	Shower and Bath: 40mm diam waste with 75mm deep seal anti syphon trap above floor to 100mm waste in floor void
W/B: 32mm diam waste with 75mm deep seal trap to wc waste.	W/B: 32mm diam waste with 75mm deep seal trap to wc waste.
WC: 100mm diam waste connection to 100mm waste in floor void to drop to below ground drain.	WC: 100mm diam waste connection to 100mm waste in floor void to drop to below ground drain in corner of Kitchen.



CAROLINE
020 2076 2511
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LONDON
0207 430 3500

WREXHAM
01978 357 588

client
Mike Proke Electrical

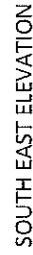
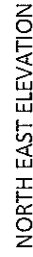
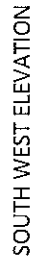
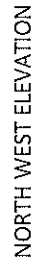
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12/15	150	CCH		
PROJECT NO	FORMING NO	PERMITS	DATE	REV
N1681	CHH-LAW-XJ01	OFF-AR-051103	P3	

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Design life measurements are carried out throughout the design stages of the project, in accordance with the company procedures and technical standards. Where necessary, thousands of samples of the components in the design have been manufactured and then destroyed, tested or used for service. Also that general rule of which a competent designer or contractor should be aware must not be overlooked. This drawing is to be read in conjunction with the Pre-Construction Information Pack and all related documents prepared in accordance with the current Construction Design and Management Regulations 2017 and all applicable construction standards in force.

EXTERNAL WALLS; MULTI STOCK FACING BRICK
WITH FLUSH POINTED NATURAL MORTAR AND
WITH BUFF COLOURED RECONSTITUTED
STONE CILLS AND HEADS TO OPENINGS.



CARDIFF 020 2076 2511
LONDON 0207 139 5500
WREXHAM 01978 357 885

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CLIENT
Miles Proulx Electrical

DRAWING TITLE
Proposed Elevations

DATE	SCALE @ 1:100	DRAWN	CHECKED	APPROVED
02-15		COH	-	
PROJECT NO. DRAWING NO. REVISIONS				
PROJECT: CHH-LAW-X-X-DEL-AR-062101				
P2				
DO NOT SCALE. Any dimensions or notes appearing on this drawing should be referred to as indicated.				
Copyright © LAWREY & LIMITED Registered Office: The Law Practice, 20000 Wilshire Blvd., Suite 2000, Los Angeles, CA 90024-3000 Phone: (310) 207-2333 Fax: (310) 207-2334				

Design life assessments are carried out throughout the design stages of the product, in accordance with the company procedures and industrial standards. Where necessary, products at various stages of the design process are subjected to destructive and non-destructive testing, subject to prior agreement with the product owner. Design and test records are maintained, subject to the same review and approval as the product itself. The company is in full compliance with the CE Marking Directive (89/339/EEC) and related documents prepared in accordance with the current Construction Products and Marketing Regulations 2007 and all applicable construction regulations as mentioned.

CARDIFF
029 2076 2511
www.lawrty.co.uk

LONDON
0207 138 3560

WREXHAM
01978 357 887

client
Miles Pryde Electrical

DATE	SCALE 0.15	DRAWING	CHECKED APPROVED
02.15	150	COH	
PROJECT NO DRAWING NO (REFERENCE)		REV	
PROJECT - CHAIRMAN - ZONE - LANE - TYPE - RULE - CLASSIFICATION			
N1681	CHH-LAW-X-X-DSC-AR-063001	P2	

DO NOT SCALE any dimensions or sizes according to drawing should be referred to be located

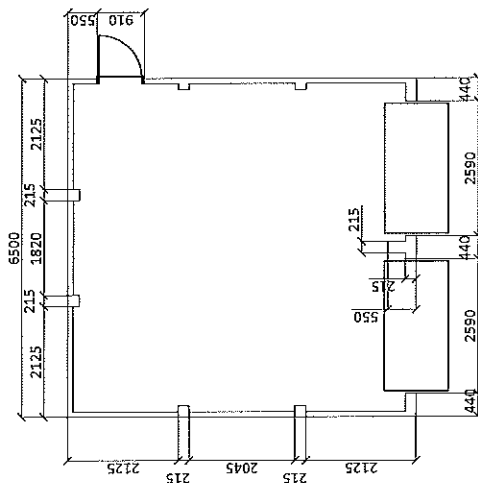
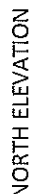
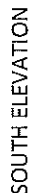
Copyright © LAMBERT LIMITED
Registered Office 17th Floor, Lambton COURT 27th Ave, No. 40, 22015, Tel: 29-5246 48

Design risk assessments are carried out throughout the design steps of this project, in accordance with the company procedures and technical manuals. Where reasonably practicable all areas of risk attributable to the design have been identified and their likelihood, frequency or extent has been estimated. The most serious risks of which a completed design or contractor shall be aware are set out below.

The following is intended to complement with the Pre-Construction Information Pack and related documents prepared in accordance with the current Construction Design Management Regulations 2013 and all applicable construction legislation as mentioned.

LEAVES, VERGES AND RAINWATER GOODS :
WHITE UPVC

EXTERNAL WALLS, MULTI STOCK FACING BRICK
WITH FLUSH POINTED NATURAL MORTAR AND
WITH BUFF COLOURED RECONSTITUTED
STONE HEADS TO OPENINGS.



FLOOR PLAN

<u>CASE NO.</u>	<u>SUBJECT</u>	<u>DATE</u>	<u>STATUS</u>
001	LEVEL OF MODEL DEFINITION (LOD)		PENDING FOR ...

[illegible]

CLINT
Wito Pryde Electrical
 www.lawry.co.uk
 CARDIFF 02929 2076 2511
 LONDON 0207 138 3560
 WREXHAM 01978 357 887

PROJECT TITLE
Replacement Dwelling, The Lodge, Crossbowell Lane, Burton

DRAWING TITLE
Proposed Garage

DATE	SCALE & S	DRAWN	CHECKED	APPROVED
10-12-15	1:100	COH	-	
PROJECT NO	DRAWING NO	PROJECT NAME	REV	
N1681	CHH-LAW-X-X-DSP-AR-910004	P1		

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 Telephone: +44 (0) 20 706 1000 Fax: +44 (0) 20 706 1001 E-mail: info@lambs.co.uk

Calculation Sheet

Project/Reference No: 026306

Sheet No: 09

Project and Description: THE LODGE CROES NOVEL
NEW DWELLING

Office: WREXHAM

Designed: IW

Checked:

LOADING		kN/m ²	
<u>ROOF: DEAD (30° PITCH ROOF)</u>			
SLATE TILES	0.60		
BATTENS	0.02		
FELT	0.02		
RAFTERS	0.10		
	<u>0.74</u>		
RAILROAD	<u>0.74</u>	=	0.86 ✓
	<u>COS 30</u>		
INSULATION		0.02	
JOISTS		0.15	
PLASTERBOARD + SKIM		<u>0.15</u>	
		<u>1.18</u>	✓
<u>IMPOSED</u>			
SNOW		<u>0.75</u>	
CEILING		<u>0.25</u>	
		<u>1.00</u>	✓
<u>FIRST FLOOR: DEAD</u>			
CHIPBOARD		0.10	
JOISTS		0.15	
INSULATION		0.02	
PLASTERBOARD + SKIM		0.15	
PARTITIONS ALLOWANCE		<u>1.00</u>	
		<u>1.42</u>	
<u>IMPOSED</u>			
RESIDENTIAL		<u>1.50</u>	✓

Calculation Sheet

Project/Reference No:	UR6306	Sheet No:	10
Project and Description:	THE LODGE CROES HOWELL NEW DWELLING	Office:	WREXHAM
		Designed:	IW
		Checked:	

LOADING (CONT.)		kg/m ²
EXTERNAL WALLS: DEAD		
100mm BRICK		2.00
PARTIAL INSULATION		0.05
100mm MEDIUM DENSE BLOCK		1.80
PLASTERBOARD + SKIM		0.10
		<u>3.95</u>
INTERNAL WALL: DEAD		
100mm BLOCK		1.80
2x PLASTERBOARD + SKIM		0.20
		<u>2.00</u>

Calculation Sheet

Project/Reference No:	VR6306	Sheet No:	11
Project and Description:	THE LODGE CROSS HOWELL NEW DWELLING	Office:	WREXHAM
		Designed:	IW
		Checked:	

KITCHEN STEEL BEAM

STEEL BEAMS REQUIRED TO SUPPORT 1ST FLOOR WALL ABOVE

CLEAR SPAN = 4.6m / - NO FLOOR LOADS

LOADING

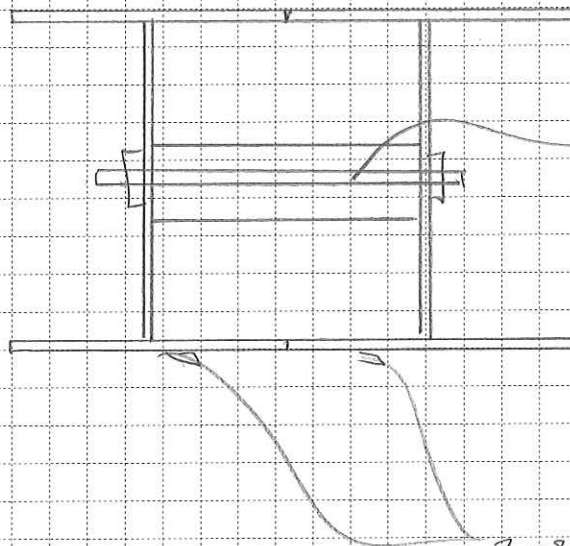
WALL : D.L = $3.95 \times 4.1m = 16.2kN/m$

PROVIDE 2N° BEAMS TO SUPPORT WIDTH OF CAVITY AND TO REDUCE BEAM DEPTH TO SIT WITHIN FLOOR CONSTRUCTION (195mm)

SEE PG-12

PROVIDE 2N° IS2x152x37 UC

DEFLECTION GOVERNS ✓



BOLT TOGETHER
WITH M10 BOLTS
@ 450 C/C WITH
SPACER BLOCKS
TO SUIT ✓

2N° IS2x152x37 UC



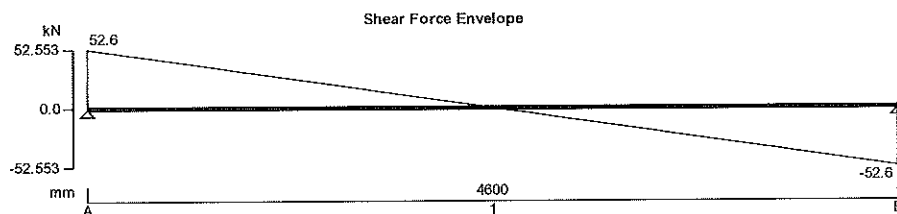
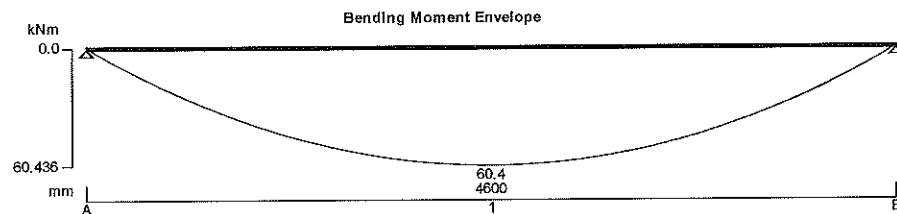
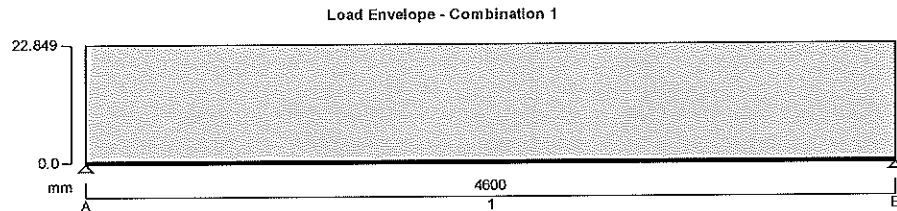
Opus International
Opus House, Yale Business Village
Wrexham Technology Park, Wrexham
LL13 7YL

Project THE LODGE CROES HOWELL				Job no. VR6306	
Calcs for KITCHEN BEAMS				Start page no./Revision 12	
Calcs by IW	Calcs date 29/06/2016	Checked by	Checked date	Approved by	Approved date

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.13



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 16.2 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Span 1

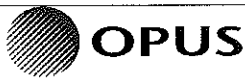
Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$



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Wrexham Technology Park, Wrexham
LL13 7YL

Project THE LODGE CROES HOWELL				Job no. VR6306	
Calcs for KITCHEN BEAMS				Start page no./Revision 13	
Calcs by IWV	Calcs date 29/06/2016	Checked by	Checked date	Approved by	Approved date

Analysis results

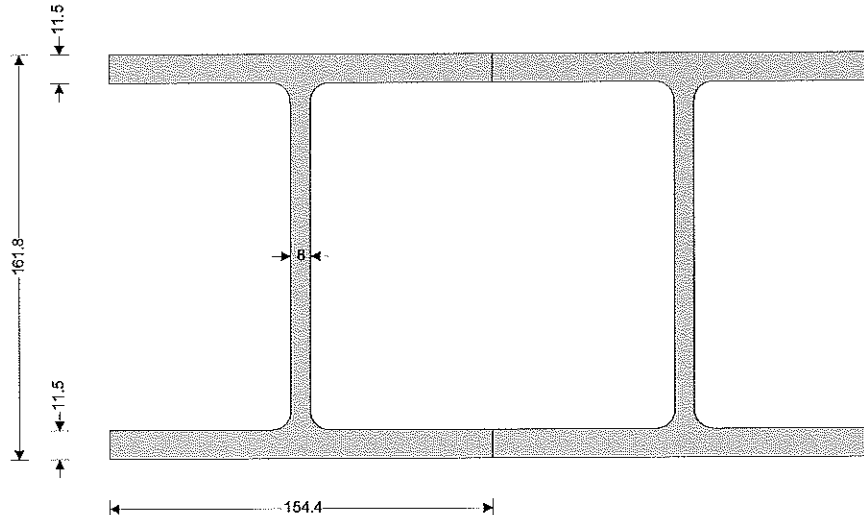
Maximum moment	$M_{\max} = 60.4 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 52.6 \text{ kN}$	$V_{\min} = -52.6 \text{ kN}$
Deflection	$\delta_{\max} = 10.6 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 52.6 \text{ kN}$	$R_{A_{\min}} = 52.6 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 38.9 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 52.6 \text{ kN}$	$R_{B_{\min}} = 52.6 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 38.9 \text{ kN}$	

Section details

Section type **2 x UKC 152x152x37 (Tata Steel Advance)**
Steel grade **S355**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	$t = \max(t_r, t_w) = 11.5 \text{ mm}$
Nominal yield strength	$f_y = 355 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 470 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT,A} = 1.000$
	$K_{LT,B} = 1.000$

Classification of cross sections - Section 5.5

$$\varepsilon = \sqrt{235 \text{ N/mm}^2 / f_y} = 0.81$$



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Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section $c = d = 123.6 \text{ mm}$
 $c / t_w = 19.0 \times \varepsilon \leq 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = 65.6 \text{ mm}$
 $c / t_f = 7.0 \times \varepsilon \leq 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = 138.8 \text{ mm}$
Shear area factor $\eta = 1.000$
 $h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 52.6 \text{ kN}$
Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1427 \text{ mm}^2$
Design shear resistance - cl 6.2.6(2) $V_{a,Rd} = V_{pl,Rd} = N \times A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 584.9 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 60.4 \text{ kNm}$
Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = N \times W_{pl,y} \times f_y / \gamma_{M0} = 219.2 \text{ kNm}$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 $k_c = 0.94$
 $C_1 = 1 / k_c^2 = 1.132$
Curvature factor $g = \sqrt{1 - (I_z / I_y)} = 0.825$
Poissons ratio $\nu = 0.3$
Shear modulus $G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$
Unrestrained length $L = 1.0 \times L_{s1} = 4600 \text{ mm}$
Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 158.9 \text{ kNm}$
Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.831$
Limiting slenderness ratio $\bar{\lambda}_{LT,0} = 0.4$
 $\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - **Lateral torsional buckling cannot be ignored**

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 b
Imperfection factor - Table 6.3 $\alpha_{LT} = 0.34$
Correction factor for rolled sections $\beta = 0.75$
LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 0.832$
LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.800$
Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.970$
Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.825$
Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times N \times W_{pl,y} \times f_y / \gamma_{M1} = 180.8 \text{ kNm}$

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads
Limiting deflection $\delta_{lim} = L_{s1} / 360 = 12.8 \text{ mm}$

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Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 10.629 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit



Calculation Sheet

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		Checked:	

BEAM END BEARING

CHECK LOAD FROM BEAM REACTIONS

SLS D.L = 38.9 kN

LOAD FROM 1ST FLOOR ABOVE

ROOF: D.L = 1.18 x 8.6m / 2 = 5.1 kN/m

I.L = (0.75 + 0.25) x 8.6m / 2 = 4.3 kN/m

FLOOR D.L = 1.42 x 4.6m / 2 = 3.3 kN/m

I.L = 1.5 x 4.6m / 2 = 3.5 kN/m

WALL D.L = 3.95 x 2.1m = 8.3 kN/m

D.L = 16.7 kN/m

I.L = 7.8 kN/m

SEE PG 17 FOR BEAM PERPENDICULAR TO WALL

PROVIDE 440W x 100 T x 215 D CONCRETE

PADSTONE TO INNER LEAF OF CAVITY



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MASONRY BEARING DESIGN

In accordance with EN1996-1-1:2005 + A1:2012, incorporating Corrigenda February 2006 and July 2009 and the UK National Annex.

Tedds calculation version 1.0.02

Masonry panel details

Panel length	$L = 14100 \text{ mm}$
Panel height	$h = 2500 \text{ mm}$
Thickness of load bearing leaf	$t = 100 \text{ mm}$
Effective height	$h_{ef} = 2500 \text{ mm}$
Effective thickness	$t_{ef} = 166 \text{ mm}$

Masonry material details

Unit type	Aggregate Concrete - Group 1
Mean compressive strength of masonry unit	$f_b = 7.3 \text{ N/mm}^2$
Specific weight of units	$\gamma = 14 \text{ kN/m}^3$
Mortar type	M4 - General Purpose
Compressive strength of mortar	$f_m = 4.0 \text{ N/mm}^2$
Compressive strength factor - Tbl. NA 4	$K = 0.75$
Characteristic compressive strength of the masonry - eq. 3.1	$f_k = K \times f_b^{0.7} \times f_m^{0.3} = 4.57 \text{ N/mm}^2$
Short term secant modulus of elasticity factor	$K_E = 1000$
Modulus of elasticity - cl.3.7.2	$E_w = K_E \times f_k = 4571 \text{ N/mm}^2$ ✓

Design compressive strength of masonry

Category of manufacturing control	Category II
Class of execution control	Class 2
Partial factor for material strength in direct or flexural compression	

$$\gamma_M = 3.00$$

Cross-sectional area of wall	$A = L \times t = 1.41 \text{ m}^2$
Design compressive strength of masonry	$f_d = f_k / \gamma_M = 1.52 \text{ N/mm}^2$

Partial safety factors for design loads

Partial safety factor for permanent load	$\gamma_{FG} = 1.35$
Partial safety factor for variable load	$\gamma_{FQ} = 1.50$

Superimposed vertical loading details

Permanent UDL at top of wall	$g_k = 16.70 \text{ kN/m}$
Variable UDL at top of wall	$q_k = 7.80 \text{ kN/m}$
Eccentricity of permanent UDL load	$e_{gu} = 0 \text{ mm}$
Eccentricity of variable UDL load	$e_{qu} = 0 \text{ mm}$

Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit	$\lambda_{lim} = 27$
Slenderness ratio	$\lambda = h_{ef} / t_{ef} = 15.1$

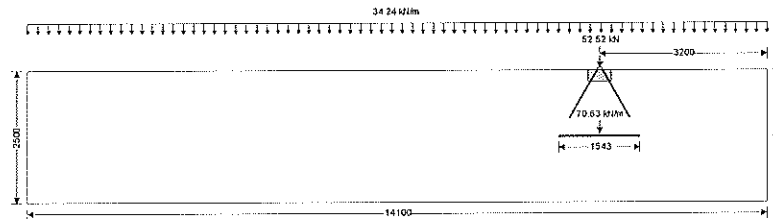
PASS - Slenderness ratio is less than slenderness limit



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Concentrated Load 1 details



Permanent concentrated load	$G_{kc1} = 38.90 \text{ kN} \checkmark$
Variable concentrated load	$Q_{kc1} = 0.00 \text{ kN}$
Eccentricity of concentrated load	$e_{c1} = 0 \text{ mm}$
Length of concentrated load	$L_{c1} = 100 \text{ mm}$
Width of concentrated load	$w_{c1} = 100 \text{ mm}$
Height of concentrated load	$h_{c1} = 2500 \text{ mm}$
Distance of load to right vertical edge	$r_{11} = 3150 \text{ mm}$
Distance of load to nearest vertical edge	$a_{11} = 3150 \text{ mm}$

Walls subjected to concentrated loads - Section 6.1.3

Eccentricity check $e_{c1} \leq t / 4$

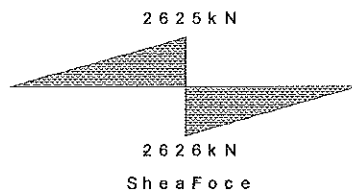
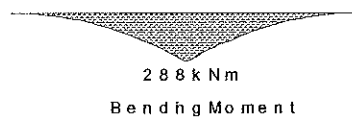
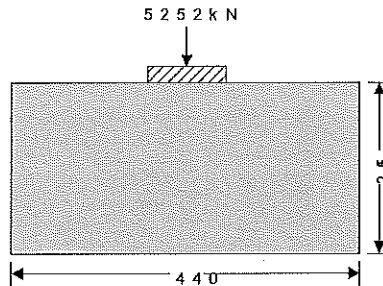
PASS - Eccentricity of load is less than $t/4$

Area of bearing	$A_{b1} = L_{c1} \times w_{c1} = 10000 \text{ mm}^2$
Effective length of bearing at mid-height	$l_{efm1} = L_{c1} + h_{c1} \times \tan(30) = 1543 \text{ mm}$
Effective bearing area	$A_{ef1} = l_{efm1} \times t = 154338 \text{ mm}^2$
Bearing area ratio check	$A_{ratio1} = \text{Min}(A_{b1} / A_{ef1}, 0.45) = 0.06$
Initial enhancement factor	$\beta_{init1} = \text{Max}((1 + 0.3 \times a_{11} / h_{c1}) \times (1.5 - 1.1 \times A_{ratio1}), 1.0) = 1.97$
Maximum enhancement factor	$\beta_{max1} = \text{Min}(1.25 + a_{11} / (2 \times h_{c1}), 1.5) = 1.50$
Enhancement factor for concentrated loads	$\beta_1 = \text{Min}(\beta_{init1}, \beta_{max1}) = 1.50$
Design value of the concentrated load	$N_{Edc1} = G_{kc1} \times \gamma_{FG} + Q_{kc1} \times \gamma_{FQ} = 52.52 \text{ kN}$
Design value concentrated load resistance	$N_{Rdc1} = \beta_1 \times A_{b1} \times f_d = 22.85 \text{ kN}$

Applied concentrated load exceeds design resistance, spreader required!

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Design of spreader beam



Type of spreader
Type of bearing onto spreader
Location of load from RHS of spreader
Length of spreader
Height of spreader
Width of spreader
Modulus of elasticity
Second moment of area
Modulus of the wall
Winkler's constant
Characteristic of the system
Classification of spreader
Krilov's functions for the spreader length

Concrete padstone

Point load

$$P_{11} = 220 \text{ mm}$$

$$L_{sp1} = 440 \text{ mm}$$

$$h_{sp1} = 215 \text{ mm}$$

$$w_{sp1} = 100 \text{ mm}$$

$$E_{sp1} = 29962 \text{ N/mm}^2$$

$$I_{sp1} = 1/12 \times w_{sp1} \times h_{sp1}^3 = 82819792 \text{ mm}^4$$

$$k_0 = E_w / h = 1.83 \text{ N/mm}^2/\text{mm}$$

$$K_{c1} = k_0 \times w_{sp1} = 182.84 \text{ N/mm/mm}$$

$$\alpha_1 = (K_{c1} / (4 \times E_{sp1} \times I_{sp1}))^{1/4} = 0.00207 \text{ mm}^{-1}$$

$$\alpha L_1 = \alpha_1 \times L_{sp1} = 0.91 \text{ Medium}$$

$$B_{\alpha 1} = 1/2 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) + \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi)) = 0.89$$

$$C_{\alpha 1} = 1/2 \times \sinh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) = 0.41$$

$$D_{\alpha 1} = 1/4 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) - \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi)) = 0.13$$

$$A_{\alpha P11} = \cosh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi) = 0.99$$

$$B_{\alpha P11} = 1/2 \times (\cosh(\alpha_1 \times P_{11}) \times \sin(180 \times \alpha_1 \times P_{11} / \pi) + \sinh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi)) = 0.46$$

Krilov's functions at the point load



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Using method of initial conditions

Initial moment of LH edge

$$M_{01} = 0 \text{ kNm}$$

Initial shear of LH edge

$$V_{01} = 0 \text{ kN}$$

Which gives

$$(4 \times \alpha_1^2 \times C_{\alpha 1} \times \delta_{01} + 4 \times \alpha_1 \times D_{\alpha 1} \times \Phi_{01}) \times E_{sp1} \times l_{sp1} - B_{\alpha P11} / \alpha_1 \times N_{Edc1} = 0.00 \text{ kNm}$$

and

$$(4 \times \alpha_1^3 \times B_{\alpha 1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{\alpha 1} \times \Phi_{01}) \times E_{sp1} \times l_{sp1} - A_{\alpha P11} \times N_{Edc1} = 0.00 \text{ kN}$$

Therefore,

Initial deflection of LH edge

$$\delta_{01} = 0.64438 \text{ mm}$$

Initial rotation of LH edge

$$\Phi_{01} = 0.000085$$

Location of maximum deflection

$$x_{def1} = 220 \text{ mm}$$

Krilov's functions at the spreader length

$$A_{\alpha xdef1} = \cosh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi) = 0.99$$

$$B_{\alpha xdef1} = 1/2 \times (\cosh(\alpha_1 \times x_{def1}) \times \sin(180 \times \alpha_1 \times x_{def1} / \pi) + \sinh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi)) = 0.46$$

Distance of point load right of loaction

$$p_{1def1} = 0 \text{ mm}$$

Krilov's functions at the spreader length

$$D_{\alpha p1def1} = 1/4 \times (\cosh(\alpha_1 \times p_{1def1}) \times \sin(180 \times \alpha_1 \times p_{1def1} / \pi) - \sinh(\alpha_1 \times p_{1def1}) \times \cos(180 \times \alpha_1 \times p_{1def1} / \pi)) = 0.00$$

Particular integral due to load

$$\delta'_1 = D_{\alpha p1def1} / \alpha_1^3 \times N_{Edc1} / (l_{sp1} \times E_{sp1}) = 0.000 \text{ mm}$$

Maximum deflection

$$\delta_{max1} = A_{\alpha xdef1} \times \delta_{01} + B_{\alpha xdef1} \times \Phi_{01} / \alpha_1 + \delta'_1 = 0.658 \text{ mm}$$

Location of maximum moment

$$x_{M1} = 220 \text{ mm}$$

Krilov's functions at the spreader length

$$C_{\alpha xM1} = 1/2 \times \sinh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) = 0.10$$

$$D_{\alpha xM1} = 1/4 \times (\cosh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) - \sinh(\alpha_1 \times x_{M1}) \times \cos(180 \times \alpha_1 \times x_{M1} / \pi)) = 0.02$$

Distance of point load right of loaction

$$p_{1M1} = 0 \text{ mm}$$

Krilov's functions at the spreader length

$$B_{\alpha p1M1} = 1/2 \times (\cosh(\alpha_1 \times p_{1M1}) \times \sin(180 \times \alpha_1 \times p_{1M1} / \pi) + \sinh(\alpha_1 \times p_{1M1}) \times \cos(180 \times \alpha_1 \times p_{1M1} / \pi)) = 0.00$$

Particular integral due to load

$$M'_1 = -B_{\alpha p1M1} / \alpha_1 \times N_{Edc1} = 0.00 \text{ kNm}$$

Maximum moment

$$M_{Edsp1} = (4 \times \alpha_1^2 \times C_{\alpha xM1} \times \delta_{01} + 4 \times \alpha_1 \times D_{\alpha xM1} \times \Phi_{01}) \times (l_{sp1} \times E_{sp1}) + M'_1 = 2.88 \text{ kNm}$$

Location of maximum shear

$$x_{V1} = 220 \text{ mm}$$

Krilov's functions at the spreader length

$$B_{\alpha xV1} = 1/2 \times (\cosh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) + \sinh(\alpha_1 \times x_{V1}) \times \cos(180 \times \alpha_1 \times x_{V1} / \pi)) = 0.46$$

$$C_{\alpha xV1} = 1/2 \times \sinh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) = 0.10$$

Distance of point load right of loaction

$$p_{1V1} = 0 \text{ mm}$$

Krilov's functions at the spreader length

$$A_{\alpha p1V1} = \cosh(\alpha_1 \times p_{1V1}) \times \cos(180 \times \alpha_1 \times p_{1V1} / \pi) = 1.00$$

Particular integral due to load

$$V'_1 = -A_{\alpha p1V1} \times N_{Edc1} = -52.51 \text{ kN}$$

Shear at concentrated point load

$$V_1 = (4 \times \alpha_1^3 \times B_{\alpha xV1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{\alpha xV1} \times \Phi_{01}) \times (l_{sp1} \times E_{sp1}) + V'_1 = -26.26 \text{ kN}$$

Maximum shear

$$V_{Edsp1} = \text{Max}(\text{Abs}(V_1), N_{Edc1} - \text{Abs}(V_1)) = 26.26 \text{ kN}$$

Maximum allowable stress under spreader

$$\sigma_{Rdsp1} = 1.5 \times f_d = 2.29 \text{ N/mm}^2$$

Maximum reaction

$$N_{Edsp1} = K_{c1} \times \delta_{max1} = 120.38 \text{ kN/m}$$

Design stress

$$\sigma_{Edsp1} = N_{Edsp1} / w_{sp1} = 1.20 \text{ N/mm}^2$$

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PASS - Design stress under spreader is less than the allowable bearing stress

Walls subjected to mainly vertical loading - Section 6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

$$e_{gmu1} = e_{gu} \times h_{c1} / (2 \times h) = 0.0 \text{ mm}$$

Eccentricity of variable UDL at mid-height below concentrated load

$$e_{qmu1} = e_{qu} \times h_{c1} / (2 \times h) = 0.0 \text{ mm}$$

Eccentricity of concentrated load at mid-height

$$e_{mc1} = e_{c1} / 2 = 0.0 \text{ mm}$$

Initial eccentricity - cl.5.5.1.1(4)

$$e_{init} = h / 450 = 5.6 \text{ mm}$$

Concentrated load at mid-height as UDL

$$N_{mc1} = N_{Edc1} / l_{efm1} = 34.03 \text{ kN/m}$$

Vertical load at mid-height

$$N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1} = 70.63 \text{ kN/m}$$

Design moment at mid-height

$$M_{Ed1} = g_k \times \gamma_{fG} \times e_{gmu1} + q_k \times \gamma_{fQ} \times e_{qmu1} + N_{mc1} \times e_{mc1} = 0.00 \text{ kNm/m}$$

Eccentricities due to loads - eq. 6.7

$$e_{m1} = \text{Abs}(M_{Ed1}) / N_{Ed1} + e_{init} = 5.6 \text{ mm}$$

Slenderness ratio limit for creep eccentricity

$$\lambda_c = 27$$

Eccentricity due to creep

$$e_{k1} = 0.0 \text{ mm}$$

Eccentricity at mid-height - eq. 6.6

$$e_{mk1} = \text{Max}(e_{m1} + e_{k1}, 0.05 \times t) = 5.6 \text{ mm}$$

From eq. G2

$$A_{t1} = 1 - 2 \times e_{mk1} / t = 0.89$$

From eq. G3

$$u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) = 0.62$$

Capacity reduction factor - eq. G1

$$\Phi_{m1} = A_{t1} \times \exp(-(u_1^2) / 2) = 0.73$$

Design vertical resistance of panel - eq.6.2

$$N_{Rd1} = \Phi_{m1} \times t \times f_d = 111.65 \text{ kN/m}$$

PASS - Design value of vertical resistance exceeds applied vertical load

Calculation Sheet

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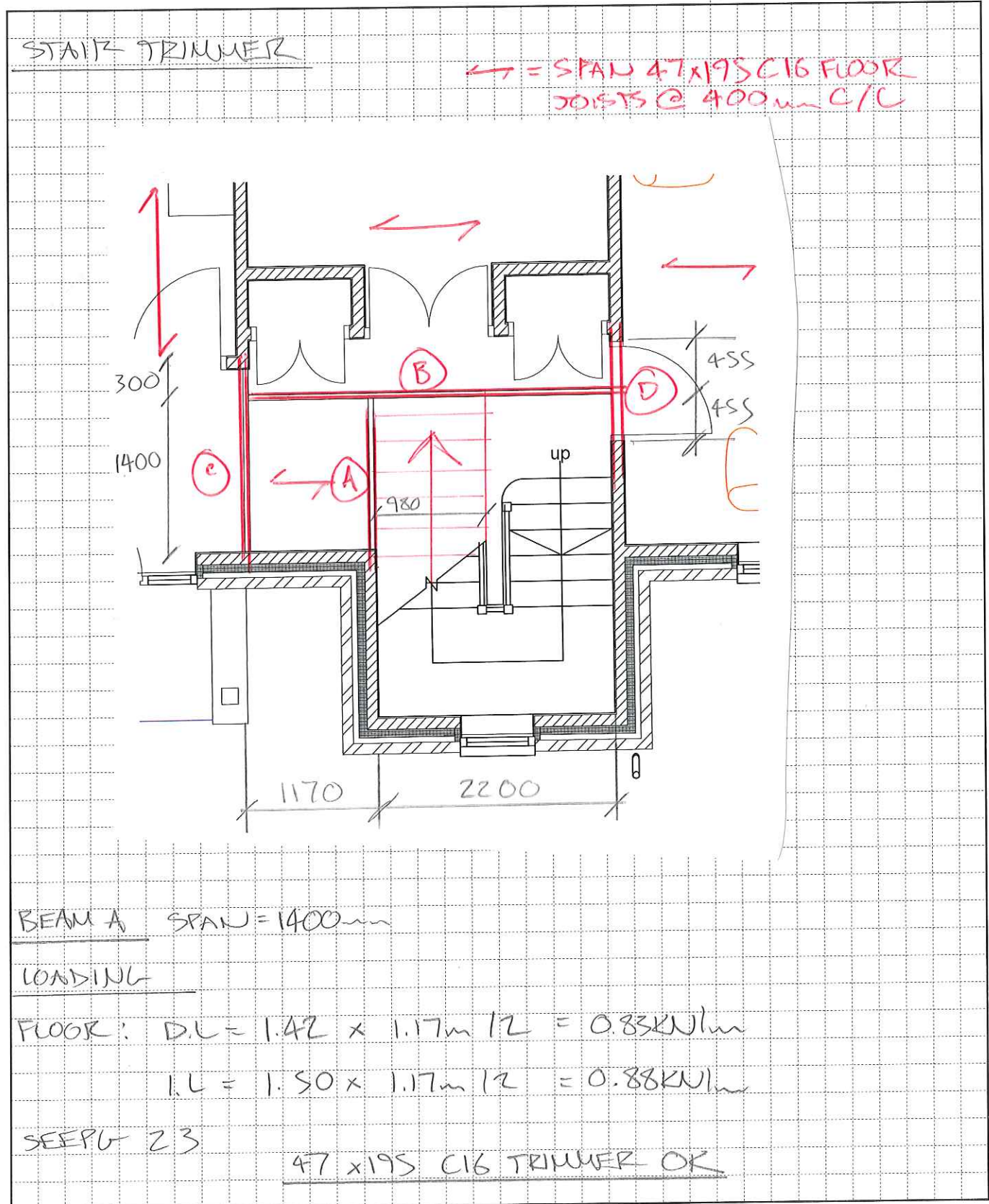
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Project and Description: THE LODGE CROES HOWELL
NEW DWELLING

Office: WREXHAM

Designed: IW

Checked:





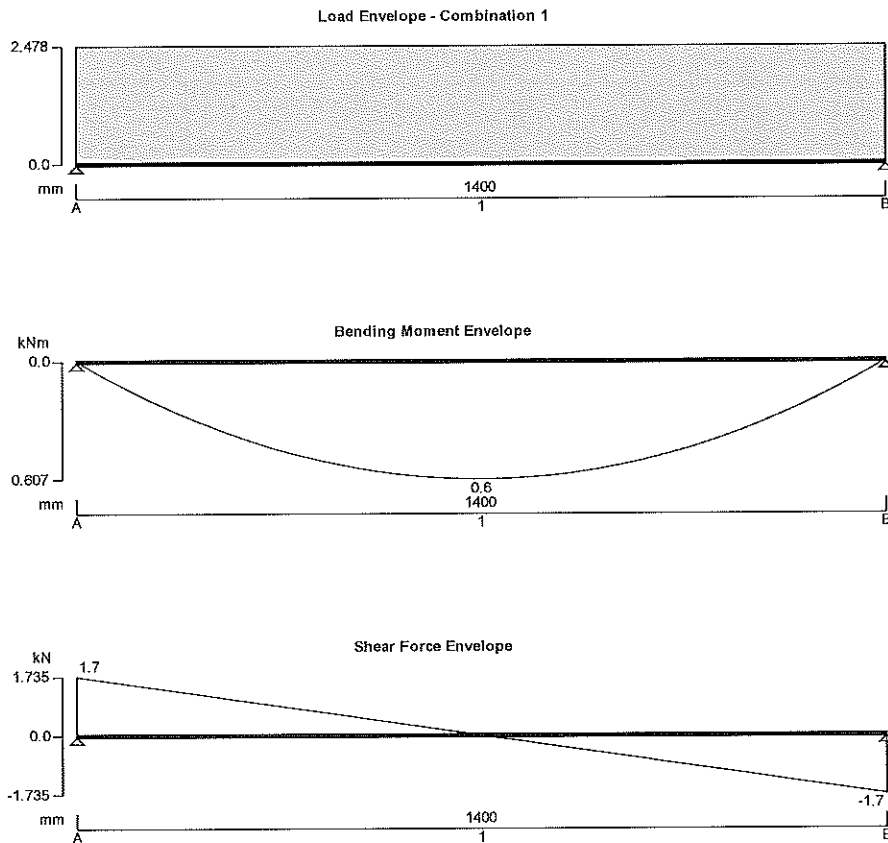
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Project				Job no.	
THE LODGE CROES HOWELL				VR6306	
Calcs for				Start page no./Revision	
STAIR TRIMMER BEAM A				23	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
IW	29/06/2016				

TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A1:2008 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 1.5.11



Applied loading

Beam loads

Permanent self weight of beam $\times 1$
Permanent full UDL 0.830 kN/m
Variable full UDL 0.880 kN/m

Load combinations

Load combination 1

Support A	Permanent $\times 1.35$ Variable $\times 1.50$
Span 1	Permanent $\times 1.35$ Variable $\times 1.50$
Support B	Permanent $\times 1.35$ Variable $\times 1.50$

Analysis results

Design moment	$M = 0.607 \text{ kNm}$	Design shear	$F = 1.735 \text{ kN}$
Total load on member	$W_{\text{tot}} = 3.469 \text{ kN}$		
Reactions at support A	$R_{A_{\text{max}}} = 1.735 \text{ kN}$	$R_{A_{\text{min}}} = 1.735 \text{ kN}$	

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Project THE LODGE CROES HOWELL				Job no. VR6306	
Calcs for STAIR TRIMMER BEAM A				Start page no./Revision 24	
Calcs by IW	Calcs date 29/06/2016	Checked by	Checked date	Approved by	Approved date

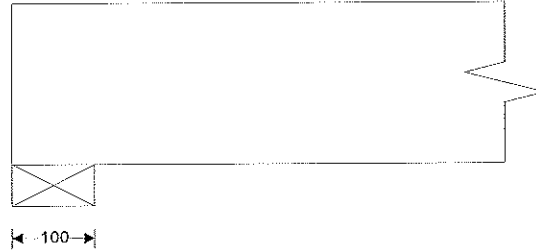
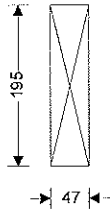
Unfactored permanent load reaction at support A $R_{A_Permanent} = 0.601 \text{ kN}$

Unfactored variable load reaction at support A $R_{A_Variable} = 0.616 \text{ kN}$

Reactions at support B $R_{B_max} = 1.735 \text{ kN}$ $R_{B_min} = 1.735 \text{ kN}$

Unfactored permanent load reaction at support B $R_{B_Permanent} = 0.601 \text{ kN}$

Unfactored variable load reaction at support B $R_{B_Variable} = 0.616 \text{ kN}$

**Timber section details**

Breadth of section	$b = 47 \text{ mm}$	Depth of section	$h = 195 \text{ mm}$
Number of sections	$N = 1$	Breadth of member	$b_b = 47 \text{ mm}$
Timber strength class	C16		

Member details

Service class of timber	1	Load duration	Long-term
Length of bearing	$L_b = 100 \text{ mm}$		

Compression perpendicular to grain - cl.6.1.4

Design compressive stress	$\sigma_{c,90,d} = 0.369 \text{ N/mm}^2$	Design compressive strength	$f_{c,90,d} = 1.185 \text{ N/mm}^2$
PASS - Design compressive strength exceeds design compressive stress at bearing			

Bending - cl 6.1.6

Design bending stress	$\sigma_{m,d} = 2.038 \text{ N/mm}^2$	Design bending strength	$f_{m,d} = 8.615 \text{ N/mm}^2$
PASS - Design bending strength exceeds design bending stress			

Shear - cl.6.1.7

Applied shear stress	$\tau_d = 0.424 \text{ N/mm}^2$	Permissible shear stress	$f_{v,d} = 1.723 \text{ N/mm}^2$
PASS - Design shear strength exceeds design shear stress			

Deflection - cl.7.2

Deflection limit	$\delta_{lim} = 5.600 \text{ mm}$	Total final deflection	$\delta_{fin} = 0.674 \text{ mm}$
PASS - Total final deflection is less than the deflection limit			

Calculation Sheet

Project/Reference No: VR6306

Sheet No: 25

Project and Description: THE LODGE CROES HOWEN
NEW DWELLING

Office: WREXHAM

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BEAM B SPAN 3370mm

SUPPORTS R₁ BEAM A + LOAD FROM STAIRCASE

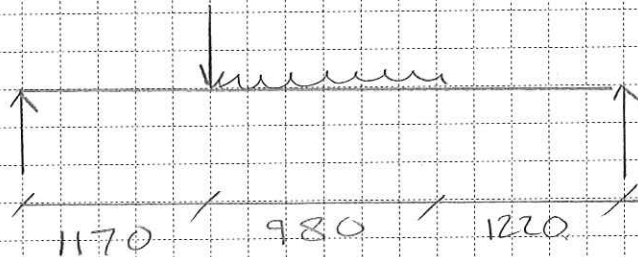
ASSUME HALF OF STAIRS LOAD BEAM /

R₁ BEAM A SLS = D.L = 0.601kN

I.L = 0.616kN

STAIR LOADS D.L = $0.42 \times 3 \text{ m} = 1.26 \text{ kN/m}$

I.L = $1.5 \times 3 \text{ m} = 4.5 \text{ kN/m}$ ✓



SEE PG 26

TO USE SAME GRADE TIMBER, USE TUBER

PITCH BEAM WITH 2N° 47x195 C16 TIMBERS

SANDWICHING 8mm x 180mm FLAT STEEL

FIXED WITH M10 COACH BOLTS @ 450mm ✓



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Project

THE LODGE CROES HOWELL

Job no.

VR6306

Calcs for

STAIR TRIMMER BEAM *AB*

Start page no./Revision

26

Calcs by

IW

Calcs date

29/06/2016

Checked by

Checked date

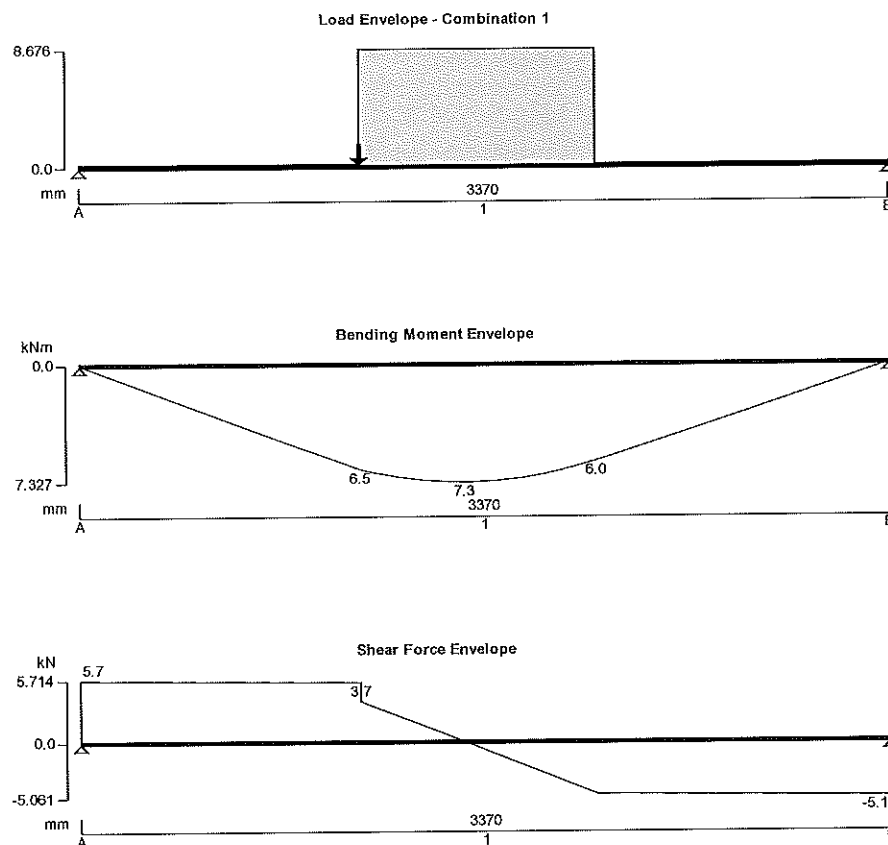
Approved by

Approved date

FLITCH BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A1:2008 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 1.5.11



Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent point load 0.601 kN at 1170 mm

Variable point load 0.616 kN at 1170 mm

Permanent partial UDL 1.260 kN/m from 1170 mm to 2150 mm

Variable partial UDL 4.500 kN/m from 1170 mm to 2150 mm

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Span 1

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$

Analysis results

Design moment

$M = 7.327$ kNm

Design shear

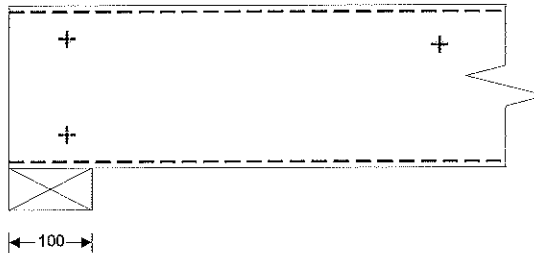
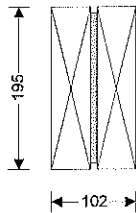
$F = 5.714$ kN



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Project THE LODGE CROES HOWELL				Job no. VR6306	
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Total load on member $W_{tot} = 10.775 \text{ kN}$
Reactions at support A $R_{A_max} = 5.714 \text{ kN}$ $R_{A_min} = 5.714 \text{ kN}$
Unfactored permanent load reaction at support A $R_{A_Permanent} = 1.300 \text{ kN}$
Unfactored variable load reaction at support A $R_{A_Variable} = 2.640 \text{ kN}$
Reactions at support B $R_{B_max} = 5.061 \text{ kN}$ $R_{B_min} = 5.061 \text{ kN}$
Unfactored permanent load reaction at support B $R_{B_Permanent} = 1.098 \text{ kN}$
Unfactored variable load reaction at support B $R_{B_Variable} = 2.386 \text{ kN}$



Timber section details

Breadth of section $b = 47 \text{ mm}$ Depth of section $h = 195 \text{ mm}$
Number of sections $N = 2$
Timber strength class **C16**

Steel section details

Breadth of steel plate $b_s = 8 \text{ mm}$ Depth of steel plate $h_s = 180 \text{ mm}$
Number of steel plates in beam $N_s = 1$ nominal yield stress f_y
 $= 275 \text{ N/mm}^2$
Bolt diameter $\phi_b = 10 \text{ mm}$

Member details

Service class of timber **1** Load duration **Long-term**
Length of bearing $L_b = 100 \text{ mm}$

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c,90,d} = 0.608 \text{ N/mm}^2$ Design compressive strength $f_{c,90,d} = 1.185 \text{ N/mm}^2$
PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design timber bending stress $\sigma_{m,t,d} = 4.461 \text{ N/mm}^2$ Design timber bending strength $f_{m,d} = 8.615 \text{ N/mm}^2$
PASS - Design timber bending strength exceeds design timber bending stress
Design steel bending stress $\sigma_{m,s,d} = 144.190 \text{ N/mm}^2$ Design steel bending strength $f_{y,d} = 275.000 \text{ N/mm}^2$
PASS - Design steel bending strength exceeds design steel bending stress

Shear - cl.6.1.7

Applied shear stress $\tau_d = 0.253 \text{ N/mm}^2$ Permissible shear stress $f_{v,d} = 1.723 \text{ N/mm}^2$
PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit $\delta_{lim} = 13.480 \text{ mm}$ Total final deflection $\delta_{fin} = 6.508 \text{ mm}$
PASS - Total final deflection is less than the deflection limit

Steel-to-timber connections - cl.8.2.3

Characteristic yield moment - exp.8.30 $M_{y,R,k} = 0.3 \text{ mm}^{0.4} \times f_{u,k} \times \phi_b^{2.6} = 47773 \text{ Nmm}$
Char.embed.strength par.to grain - exp.8.32 $f_{h,0,k} = 0.082e9 \text{ m/sec}^2 \times (1 \text{ mm} - 0.01 \times \phi_b) \times \rho_k = 22.878 \text{ N/mm}^2$
 $k_{90} = 1.35 + 0.015 \times \phi_b / 1 \text{ mm} = 1.500$



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Char.embed.strength perp.to grain - exp.8.31

$$f_{h,90,k} = f_{h,0,k} / k_{90} = 15.252 \text{ N/mm}^2$$

Thickness limit for thin steel plates

$$b_{s,thin} = \phi_b / 2 = 5 \text{ mm}$$

Thickness limit for thick steel plates

$$b_{s,thk} = \phi_b = 10 \text{ mm}$$

Characteristic load-carrying capacity for a plate of any thickness as the central member in double shear - exp.8.11

$$F_{v,Rk,f} = f_{h,k} \times b \times \phi_b = 7.168 \text{ kN}$$

$$F_{v,Rk,g} = f_{h,k} \times b \times \phi_b \times (\sqrt{2 + 4 \times M_{y,Rk} / (f_{h,k} \times \phi_b \times b^2)}) - 1 = 4.317 \text{ kN}$$

$$F_{v,Rk,h} = 2.3 \times \sqrt{M_{y,Rk} \times f_{h,k} \times \phi_b} = 6.208 \text{ kN}$$

$$F_{v,Rk} = \text{Min}(F_{v,Rk,f}, F_{v,Rk,g}, F_{v,Rk,h}) = 4.317 \text{ kN}$$

Flitch plate bolting requirements

Total load on member $W_{tot} = 10.775 \text{ kN}$

Total load taken by steel $W_s = 9.160 \text{ kN}$

Characteristic bolt capacity $F_{v,Rk} = 4.317 \text{ kN}$

Number of interfaces $N_{int} = 2$

Bolts required at supports $N_{be} = 2$

Bolts required to beam length $N_{bl} = 1.061$

Limiting bolt spacing $S_{limit} = 488 \text{ mm}$

Maximum bolt spacing $S_{max} = 450 \text{ mm}$

Provide a minimum of 2 No.10 mm diameter bolts at each support

Provide 10 mm diameter bolts at maximum 450 mm centres staggered 50 mm alternately above and below the centre line

Minimum bolt spacings - cl.8.5

Minimum end spacing $S_{end} = 70 \text{ mm}$

Minimum edge spacing $S_{edge} = 40 \text{ mm}$

Minimum bolt spacing $S_{bolt} = 40 \text{ mm}$

Minimum washer diameter $\phi_w = 30 \text{ mm}$

Minimum washer thickness $t_w = 3 \text{ mm}$

Calculation Sheet

Project/Reference No: UP6306

Sheet No: 29

Project and Description: THE LODGE CROSS HOWELL
NEW DWELLING

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BEAM C SPAN 1700mm

SUPPORTS R_A FROM BEAM B = SLS DL = 13kN

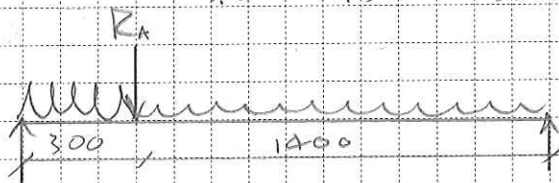
I.L = 2.64kN

PARTIAL UDL $0m-1.4m$ D.L = 0.83kN/m

I.L = 0.88kN/m ✓

PARTIAL UDL $1.4m-1.7m$ D.L = $1.42 \times 3.35m/2 = 2.38kN/m$

I.L = $1.50 \times 3.35m/2 = 2.51kN/m$ /



PG-30 - USE 2N° 47 x 195 C16 JOISTS BATED

TOGETHER WITH MID COACH BAYS @ 4500 C



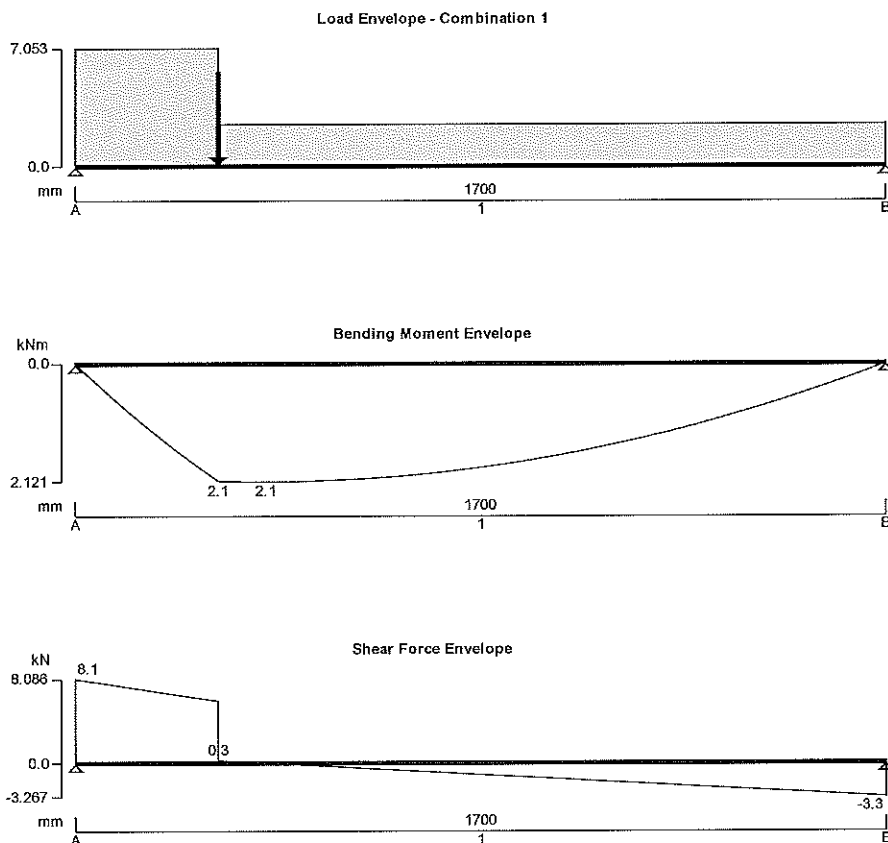
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Project THE LODGE CROES HOWELL				Job no. VR6306	
Calcs for STAIR TRIMMER BEAM C				Start page no./Revision 30	
Calcs by IW	Calcs date 29/06/2016	Checked by	Checked date	Approved by	Approved date

TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A1:2008 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 1.5.11



Applied loading

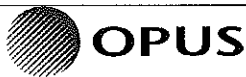
Beam loads

Permanent self weight of beam $\times 1$
Permanent point load 1.300 kN at 300 mm
Variable point load 2.640 kN at 300 mm
Permanent partial UDL 0.830 kN/m from 300 mm to 1700 mm
Variable partial UDL 0.880 kN/m from 300 mm to 1700 mm
Permanent partial UDL 2.380 kN/m from 0 mm to 300 mm
Variable partial UDL 2.510 kN/m from 0 mm to 300 mm

Load combinations

Load combination 1

Support A	Permanent $\times 1.35$ Variable $\times 1.50$
Span 1	Permanent $\times 1.35$ Variable $\times 1.50$
Support B	Permanent $\times 1.35$ Variable $\times 1.50$

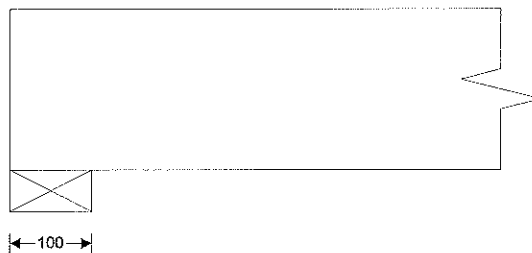
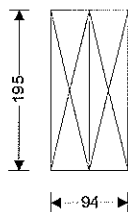


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Project THE LODGE CROES HOWELL				Job no. VR6306	
Calcs for STAIR TRIMMER BEAM C				Start page no./Revision 31	
Calcs by IW	Calcs date 29/06/2016	Checked by	Checked date	Approved by	Approved date

Analysis results

Design moment	$M = 2.121 \text{ kNm}$	Design shear	$F = 8.086 \text{ kN}$
Total load on member	$W_{\text{tot}} = 11.353 \text{ kN}$		
Reactions at support A	$R_{A_{\text{max}}} = 8.086 \text{ kN}$	$R_{A_{\text{min}}} = 8.086 \text{ kN}$	
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 2.247 \text{ kN}$		
Unfactored variable load reaction at support A	$R_{A_{\text{Variable}}} = 3.368 \text{ kN}$		
Reactions at support B	$R_{B_{\text{max}}} = 3.267 \text{ kN}$	$R_{B_{\text{min}}} = 3.267 \text{ kN}$	
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 1.023 \text{ kN}$		
Unfactored variable load reaction at support B	$R_{B_{\text{Variable}}} = 1.257 \text{ kN}$		



Timber section details

Breadth of section	$b = 47 \text{ mm}$	Depth of section	$h = 195 \text{ mm}$
Number of sections	$N = 2$	Breadth of member	$b_b = 94 \text{ mm}$
Timber strength class	C16		

Member details

Service class of timber	1	Load duration	Long-term
Length of bearing	$L_b = 100 \text{ mm}$		

Compression perpendicular to grain - cl.6.1.4

Design compressive stress	$\sigma_{c,90,d} = 0.860 \text{ N/mm}^2$	Design compressive strength	$f_{c,90,d} = 1.185 \text{ N/mm}^2$
PASS - Design compressive strength exceeds design compressive stress at bearing			

Bending - cl 6.1.6

Design bending stress	$\sigma_{m,d} = 3.561 \text{ N/mm}^2$	Design bending strength	$f_{m,d} = 8.615 \text{ N/mm}^2$
PASS - Design bending strength exceeds design bending stress			

Shear - cl.6.1.7

Applied shear stress	$\tau_d = 0.988 \text{ N/mm}^2$	Permissible shear stress	$f_{v,d} = 1.723 \text{ N/mm}^2$
PASS - Design shear strength exceeds design shear stress			

Deflection - cl.7.2

Deflection limit	$\delta_{\text{lim}} = 6.800 \text{ mm}$	Total final deflection	$\delta_{\text{fin}} = 1.501 \text{ mm}$
PASS - Total final deflection is less than the deflection limit			

✓

Calculation Sheet

Project/Reference No: UZ6306

Sheet No: 32

Project and Description: THE LODGE CROSS HOWELL
NEW DWELLING

Office: WREXHAM

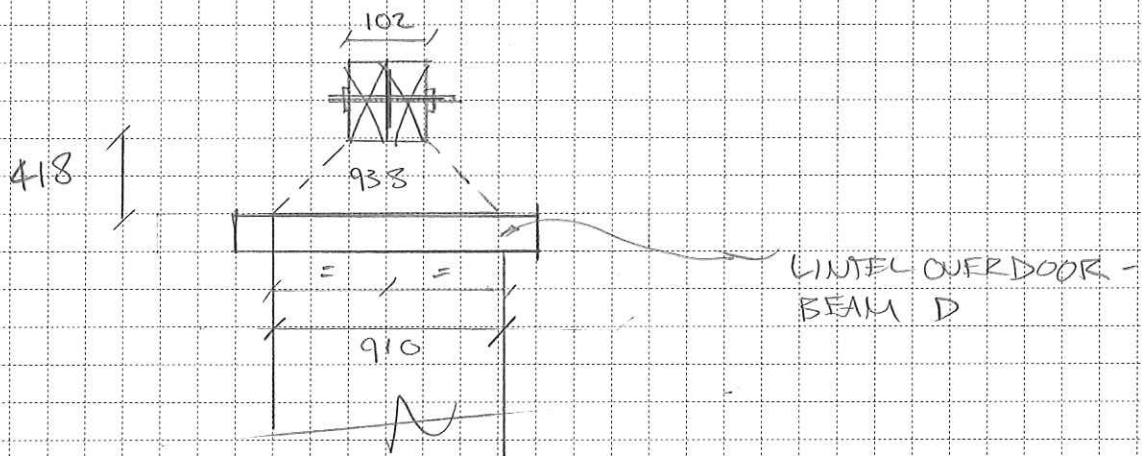
Designed: IW

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BEAM D SPAN 910mm

BEAM D ACTS AS A LINTEL OVER DOOR WAY

END OF BEAM B SITS ABOVE WITHIN FLOOR VOID AS BELOW:



AS 45° SPREAD FROM POINT LOAD FALLS OUTSIDE OF OPENING, LOAD FROM REACTION CAN BE REDUCED TO AN EQUIVALENT LINE LOAD

R_B BEAM B DL = 1.098 kN / 0.938 m = 1.17 kN/m UDL

IL = 2.386 kN / 0.938 m = 2.55 kN/m UDL

ADDITIONAL LOADING FROM FLOOR:

$$D.L = 1.42 \times \left(\frac{3.35 + 3.66}{2} \right) = 4.98 \text{ kN/m}$$

$$D.L = 1.5 \times \left(\frac{3.35 + 3.66}{2} \right) = 5.26 \text{ kN/m}$$

∴ TOTAL SLS LOAD ON LINTEL = 13.96 kN/m × 0.91 m = 12.7 kN SLS

CATNIC BSD100 BOX LINTEL SUITABLE FOR

LOAD FROM TRIMMER ABOVE

(SWL @ 900mm = 19 kN)

INTERNAL WALLS INTERNAL PARTITION AND LOADBEARING WALL LINTELS

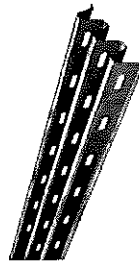
For use in internal partition and loadbearing walls 75mm, 100mm and 140mm.

Internal wall lintels

Catnic lintels for internal partitions and loadbearing walls are available in either 'corrugated', 'channel' or 'box section' to accommodate different loads and openings.

1 Corrugated

For use in solid or block walls.



**CN92
and CN102**

Offers a cost effective solution for extra light duty loads.

Suitable for nominal domestic loading.

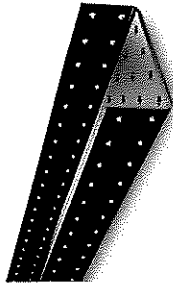
- The SWL (safe working load) is based on the total UDL (uniform distributed load) over maximum span using 75mm end bearings for CN92 and CN102.

Benefits

- Integral plaster key
Lozenge shaped staggered holes to sides of corrugated profile

2 Channel

For use in solid or block walls.



CN100

Offers a cost effective solution for light duty loads.

As corrugated lintel plus:
Suitable for masonry/timber floor loads.

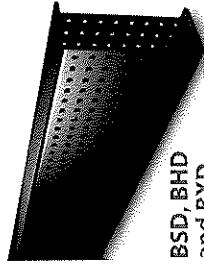
- Direct floor or roof load
- Supports concrete floor loads
- Supports point loads e.g. steel beams

Benefits

- Duplex Corrosion Protection System
Ensures optimum durability and longevity
- Integral plaster key
With holes to all sides of channel profile

3 Classic Box

For use in solid or block walls.



**BSD, BHD
and BXD**

Offers a cost effective solution for standard duty and heavy duty loads.

Universal application caters for all loading conditions:

- Direct floor or roof load
- Supports concrete floor loads
- Supports point loads e.g. steel beams

Benefits

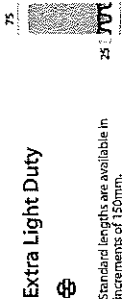
- Duplex Corrosion Protection System
Ensures optimum durability and longevity
- Integral plaster key
With staggered holes to three sides of box profile

INTERNAL WALL LINTELS

**CN
& BOX
75-140mm
INTERIOR
WALL**

75mm and 100mm Interior Solid Walls

Extra Light Duty



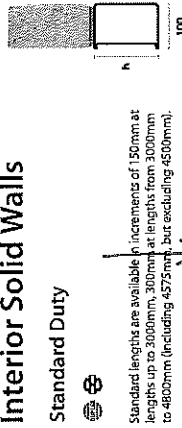
CN92

Standard lengths (mm)	1050-1200
SWL (kN)	7
Weight (kg/m)	1.2
Nominal height h (mm)	25

Notes: When using CN92 and CN102 lintels normal building practice should be observed. In this case of blockwork should be laid on the lintel and the mortar allowed to harden for at least 24 hours before additional loads are applied.

100mm Interior Solid Walls

Standard Duty



CN102

Standard lengths (mm)	1050-1200
SWL (kN)	7
Weight (kg/m)	1.8
Nominal height h (mm)	25

100mm Interior Solid Walls

Light Duty



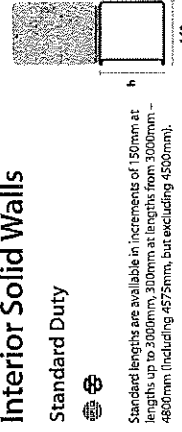
CN100

Standard lengths (mm)	1050-1200
SWL (kN)	10
Weight (kg/m)	3.7
Nominal height h (mm)	30

* Not suitable for floor loads

140mm Interior Solid Walls

Standard Duty



BSD140

Standard lengths (mm)	1050-1200
SWL (kN)	19
Weight (kg/m)	6.9
Nominal height h (mm)	143

Heavy Duty

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Extra Heavy Duty

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Standard lengths (mm)	1050-1500
SWL (kN)	47
Weight (kg/m)	10.9
Nominal height h (mm)	143

Standard lengths (mm)	1050-1500
SWL (kN)	47
Weight (kg/m)	15.7
Nominal height h (mm)	219

Standard lengths (mm)	1050-1500
SWL (kN)	47
Weight (kg/m)	59
Nominal height h (mm)	1650-2700

Standard lengths (mm)	1050-1500
SWL (kN)	47
Weight (kg/m)	59
Nominal height h (mm)	1650-2700

Calculation Sheet

Project/Reference No: UP6306

Sheet No: 34

Project and Description: THE LODGE CROES HOWELL
NEW DWELLING

Office: WREXHAM

Designed: LW

Checked:

SUB STRUCTURE DESIGN

IN VIEW OF ANY ON SITE GROUND INVESTIGATION, ONLINE BRITISH GEOLOGICAL SOCIETY (BGS) BOREHOLE SCAN VIEWER HAS BEEN USED TO DETERMINE LOCAL GROUND CONDITIONS

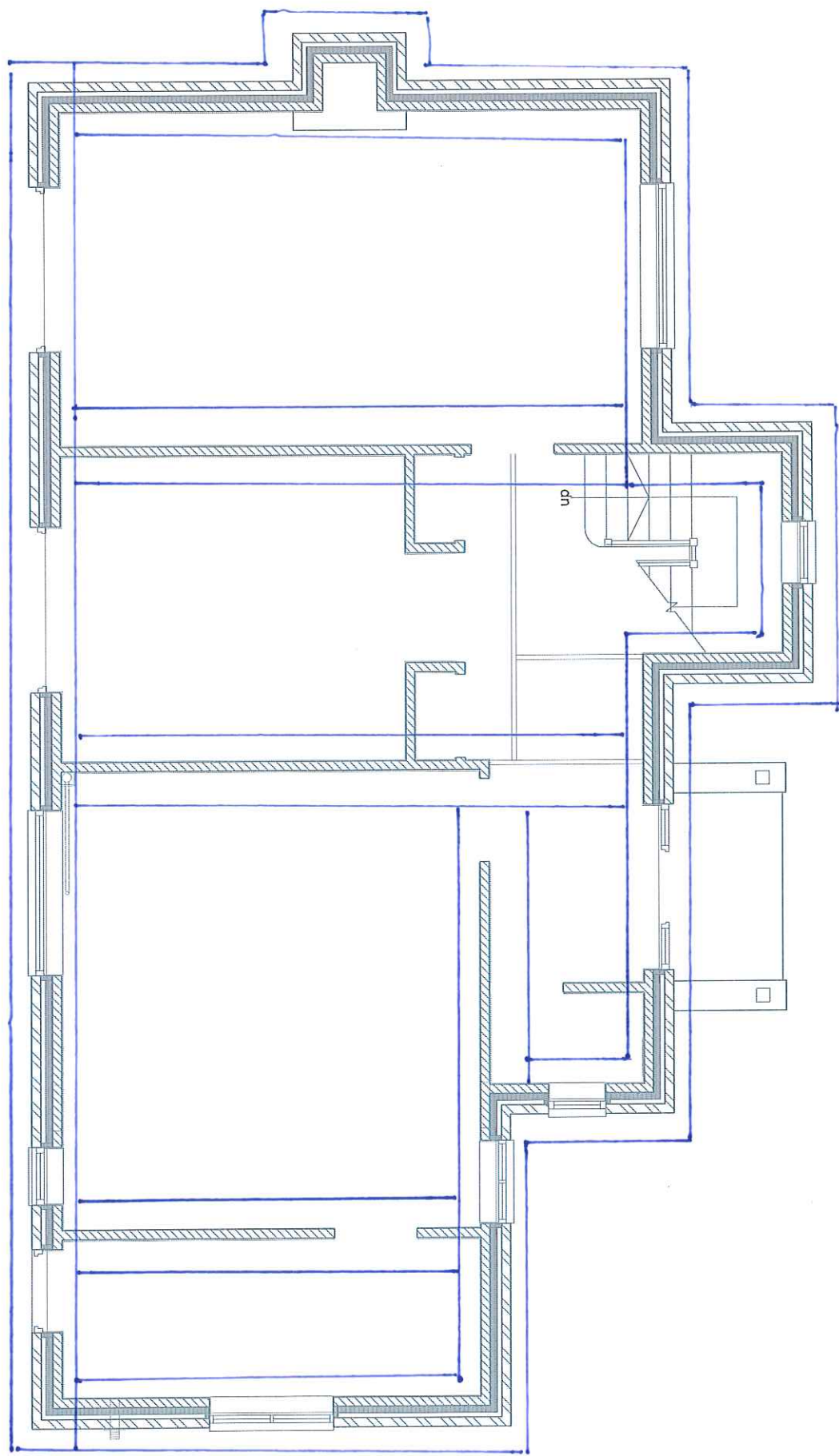
NEARBY BOREHOLES INDICATE AN OVERLYING LAYER OF GRAVEL + SANDS APPROX 2m THICK OVER CLAY

SITE ALSO FALLS WITHIN AREA REQUIRING FULL RADON PROTECTION

∴ IT IS PROPOSED TO UNDERTAKE THE FOLLOWING DESIGN:

- SHALLOW STRIP FOUNDATIONS
- 'GROUND BEARING' INSITU RC SLAB WITH RADON SUMPS

FOR PROPOSED FOUNDATION LAYOUT SEE PG 35



Calculation Sheet

Project/Reference No: UR6306

Sheet No: 36

Project and Description: THE LODGE CROESNAUEN

Office: WREXHAM

Designed: IW

Checked:

GROUND FLOOR SLAB

'GROUND BEARING' RC SLAB TO BE UTILISED ON SITE

DUE TO FULL RADON REQUIREMENTS, SLAB NEEDS TO BE SELF SUPPORTING SO THAT RADON PROTECTION BELOW IS NOT DAMAGED

∴ DESIGN SLAB TO SPAN BETWEEN SUB FLOOR WALL CONSTRUCTION

DESIGN SLAB AS 2WAY SPANNING

CHARACTERISTIC LOADS - D.L = SAY 0.5 kN/m^2 FINISHES
 5.0 kN/m^2 SLAB (200THK)

I.L = 1.5 kN/m^2 RESIDENTIAL

FOR DESIGN SEE PG 37

PROVIDE 200mm THICK 2WAY SPANNING

SLAB WITH 1 LAYER A393 MESH TOP+BTM

**OPUS**

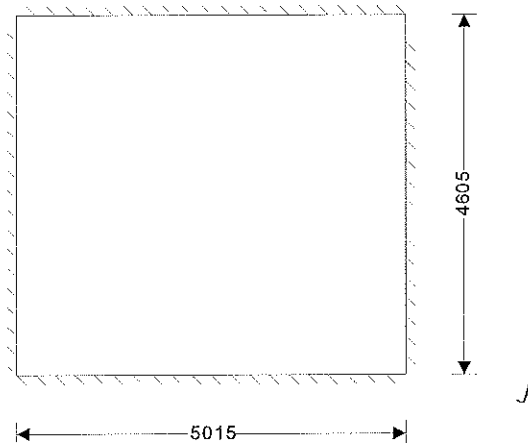
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Project THE LODGE CROES HOWELL				Job no. VR6306	
Calcs for FLOOR SLAB DESIGN				Start page no./Revision 37	
Calcs by IW	Calcs date 30/06/2016	Checked by	Checked date	Approved by	Approved date

RC SLAB DESIGN

In accordance with EN1992-1-1:2004 incorporating corrigendum January 2008 and the UK national annex

Tedds calculation version 1.0.12

**Slab definition**

Type of slab	Two way spanning with restrained edges
Overall slab depth	$h = 200 \text{ mm}$
Shorter effective span of panel	$l_x = 4605 \text{ mm}$
Longer effective span of panel	$l_y = 5015 \text{ mm}$
Support conditions	Four edges discontinuous

Bottom outer layer of reinforcement	Short span direction
-------------------------------------	-----------------------------

Loading

Characteristic permanent action	$G_k = 5.5 \text{ kN/m}^2$
Characteristic variable action	$Q_k = 1.5 \text{ kN/m}^2$
Partial factor for permanent action	$\gamma_G = 1.35$
Partial factor for variable action	$\gamma_Q = 1.50$
Quasi-permanent value of variable action	$\psi_2 = 0.30$
Design ultimate load	$q = \gamma_G \times G_k + \gamma_Q \times Q_k = 9.7 \text{ kN/m}^2$
Quasi-permanent load	$q_{SLS} = 1.0 \times G_k + \psi_2 \times Q_k = 6.0 \text{ kN/m}^2$

Concrete properties

Concrete strength class	C28/35
Characteristic cylinder strength	$f_{ck} = 28 \text{ N/mm}^2$
Partial factor (Table 2.1N)	$\gamma_C = 1.50$
Compressive strength factor (cl. 3.1.6)	$\alpha_{cc} = 0.85$
Design compressive strength (cl. 3.1.6)	$f_{cd} = 15.9 \text{ N/mm}^2$
Mean axial tensile strength (Table 3.1)	$f_{ctm} = 0.30 \text{ N/mm}^2 \times (f_{ck} / 1 \text{ N/mm}^2)^{2/3} = 2.8 \text{ N/mm}^2$
Maximum aggregate size	$d_g = 20 \text{ mm}$

Reinforcement properties

Characteristic yield strength	$f_{yk} = 500 \text{ N/mm}^2$
Partial factor (Table 2.1N)	$\gamma_S = 1.15$



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Design yield strength (fig. 3.8)

$$f_{yd} = f_{yk} / \gamma_s = 434.8 \text{ N/mm}^2$$

Concrete cover to reinforcement

Nominal cover to outer bottom reinforcement

$$C_{nom_b} = 50 \text{ mm}$$

Fire resistance period to bottom of slab

$$R_{blm} = 30 \text{ min}$$

Axis distance to bottom reinf (Table 5.8)

$$a_{fl_b} = 10 \text{ mm}$$

Min. btm cover requirement with regard to bond

$$C_{min_b_b} = 10 \text{ mm}$$

Reinforcement fabrication

Not subject to QA system

Cover allowance for deviation

$$\Delta C_{dev} = 10 \text{ mm}$$

Min. required nominal cover to bottom reinf

$$C_{nom_b_min} = 20.0 \text{ mm}$$

PASS - There is sufficient cover to the bottom reinforcement

Reinforcement design at midspan in short span direction (cl.6.1)

Bending moment coefficient

$$\beta_{sx_p} = 0.0639$$

Design bending moment

$$M_{x_p} = \beta_{sx_p} \times q \times l_x^2 = 13.1 \text{ kNm/m}$$

Reinforcement provided

10 mm dia. bars at 200 mm centres

Area provided

$$A_{sx_p} = 393 \text{ mm}^2/\text{m}$$

Effective depth to tension reinforcement

$$d_{x_p} = h - C_{nom_b} - \phi_{x_p} / 2 = 145.0 \text{ mm}$$

K factor

$$K = M_{x_p} / (b \times d_{x_p}^2 \times f_{ck}) = 0.022$$

Redistribution ratio

$$\delta = 1.0$$

K' factor

$$K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 = 0.208$$

K < K' - Compression reinforcement is not required

Lever arm

$$z = \min(0.95 \times d_{x_p}, d_{x_p} / 2 \times (1 + (1 - 3.53 \times K)^{0.5})) = 137.8 \text{ mm}$$

Area of reinforcement required for bending

$$A_{sx_p_m} = M_{x_p} / (f_{yd} \times z) = 219 \text{ mm}^2/\text{m}$$

Minimum area of reinforcement required

$$A_{sx_p_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{x_p}, 0.0013 \times b \times d_{x_p}) = 209 \text{ mm}^2/\text{m}$$

Area of reinforcement required

$$A_{sx_p_req} = \max(A_{sx_p_m}, A_{sx_p_min}) = 219 \text{ mm}^2/\text{m}$$

PASS - Area of reinforcement provided exceeds area required

Check reinforcement spacing

Reinforcement service stress

$$\sigma_{sx_p} = (f_{yk} / \gamma_s) \times \min((A_{sx_p_m}/A_{sx_p}), 1.0) \times q_{SLS} / q = 149.1 \text{ N/mm}^2$$

Maximum allowable spacing (Table 7.3N)

$$s_{max_x_p} = 300 \text{ mm}$$

Actual bar spacing

$$s_{x_p} = 200 \text{ mm}$$

PASS - The reinforcement spacing is acceptable

Reinforcement design at midspan in long span direction (cl.6.1)

Bending moment coefficient

$$\beta_{sy_p} = 0.0560$$

Design bending moment

$$M_{y_p} = \beta_{sy_p} \times q \times l_y^2 = 11.5 \text{ kNm/m}$$

Reinforcement provided

10 mm dia. bars at 200 mm centres

Area provided

$$A_{sy_p} = 393 \text{ mm}^2/\text{m}$$

Effective depth to tension reinforcement

$$d_{y_p} = h - C_{nom_b} - \phi_{x_p} - \phi_{y_p} / 2 = 135.0 \text{ mm}$$

K factor

$$K = M_{y_p} / (b \times d_{y_p}^2 \times f_{ck}) = 0.023$$

Redistribution ratio

$$\delta = 1.0$$

K' factor

$$K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 = 0.208$$

K < K' - Compression reinforcement is not required

Lever arm

$$z = \min(0.95 \times d_{y_p}, d_{y_p} / 2 \times (1 + (1 - 3.53 \times K)^{0.5})) = 128.3 \text{ mm}$$

Area of reinforcement required for bending

$$A_{sy_p_m} = M_{y_p} / (f_{yd} \times z) = 206 \text{ mm}^2/\text{m}$$

Minimum area of reinforcement required

$$A_{sy_p_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{y_p}, 0.0013 \times b \times d_{y_p}) = 194 \text{ mm}^2/\text{m}$$

Area of reinforcement required

$$A_{sy_p_req} = \max(A_{sy_p_m}, A_{sy_p_min}) = 206 \text{ mm}^2/\text{m}$$

PASS - Area of reinforcement provided exceeds area required



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Check reinforcement spacing

Reinforcement service stress

$$\sigma_{sy_p} = (f_{yk} / \gamma_s) \times \min((A_{sy_p_m} / A_{sy_p}), 1.0) \times q_{SLS} / q = 140.3 \text{ N/mm}^2$$

Maximum allowable spacing (Table 7.3N)

$$s_{max_y_p} = 300 \text{ mm}$$

Actual bar spacing

$$s_{y_p} = 200 \text{ mm}$$

PASS - The reinforcement spacing is acceptable

Shear capacity check at short span discontinuous support

Shear force

$$V_{x_d} = q \times l_x / 2 = 22.3 \text{ kN/m}$$

Reinforcement provided

$$10 \text{ mm dia. bars at } 200 \text{ mm centres}$$

Area provided

$$A_{sx_d} = 393 \text{ mm}^2/\text{m}$$

Effective depth

$$d_{x_d} = h - c_{nom_b} - \phi_{x_d} / 2 = 145.0 \text{ mm}$$

Effective depth factor

$$k = \min(2.0, 1 + (200 \text{ mm} / d_{x_d})^{0.5}) = 2.000$$

Reinforcement ratio

$$\rho_l = \min(0.02, A_{sx_d} / (b \times d_{x_d})) = 0.0027$$

Minimum shear resistance

$$V_{Rd,c_min} = 0.035 \text{ N/mm}^2 \times k^{1.5} \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times b \times d_{x_d}$$

$$V_{Rd,c_min} = 76.0 \text{ kN/m}$$

Shear resistance

$$V_{Rd,c_x_d} = \max(V_{Rd,c_min}, 0.18 \text{ N/mm}^2 / \gamma_c \times k \times (100 \times \rho_l \times (f_{ck} / 1 \text{ N/mm}^2))^{0.333} \times b \times d_{x_d})$$

$$V_{Rd,c_x_d} = 76.0 \text{ kN/m}$$

PASS - Shear capacity is adequate (0.293)

Shear capacity check at long span discontinuous support

Shear force

$$V_{y_d} = q \times l_x / 2 = 22.3 \text{ kN/m}$$

Reinforcement provided

$$10 \text{ mm dia. bars at } 200 \text{ mm centres}$$

Area provided

$$A_{sy_d} = 393 \text{ mm}^2/\text{m}$$

Effective depth

$$d_{y_d} = h - c_{nom_b} - \phi_{x_p} - \phi_{y_d} / 2 = 135.0 \text{ mm}$$

Effective depth factor

$$k = \min(2.0, 1 + (200 \text{ mm} / d_{y_d})^{0.5}) = 2.000$$

Reinforcement ratio

$$\rho_l = \min(0.02, A_{sy_d} / (b \times d_{y_d})) = 0.0029$$

Minimum shear resistance

$$V_{Rd,c_min} = 0.035 \text{ N/mm}^2 \times k^{1.5} \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times b \times d_{y_d}$$

$$V_{Rd,c_min} = 70.7 \text{ kN/m}$$

Shear resistance

$$V_{Rd,c_y_d} = \max(V_{Rd,c_min}, 0.18 \text{ N/mm}^2 / \gamma_c \times k \times (100 \times \rho_l \times (f_{ck} / 1 \text{ N/mm}^2))^{0.333} \times b \times d_{y_d})$$

$$V_{Rd,c_y_d} = 70.7 \text{ kN/m}$$

PASS - Shear capacity is adequate (0.315)

Basic span-to-depth deflection ratio check (cl. 7.4.2)

Reference reinforcement ratio

$$\rho_0 = (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / 1000 = 0.0053$$

Required tension reinforcement ratio

$$\rho = \max(0.0035, A_{sx_p_req} / (b \times d_{x_p})) = 0.0035$$

Required compression reinforcement ratio

$$\rho' = A_{sxc_p_req} / (b \times d_{x_p}) = 0.0000$$

Structural system factor (Table 7.4N)

$$K_\delta = 1.0$$

Basic limit span-to-depth ratio $ratio_{lim_x_bas} = K_\delta \times [11 + 1.5 \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times \rho_0 / \rho + 3.2 \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times (\rho_0 / \rho - 1)^{1.5}]$

(Exp. 7.16)

$$ratio_{lim_x_bas} = 29.20$$

Mod span-to-depth ratio limit $ratio_{lim_x} = \min(40 \times K_\delta, \min(1.5, (500 \text{ N/mm}^2 / f_{yk}) \times (A_{sx_p} / A_{sx_p_m})) \times ratio_{lim_x_bas}) = 40.00$

Actual span-to-eff. depth ratio

$$ratio_{act_x} = l_x / d_{x_p} = 31.76$$

PASS - Actual span-to-effective depth ratio is acceptable

Reinforcement sketch

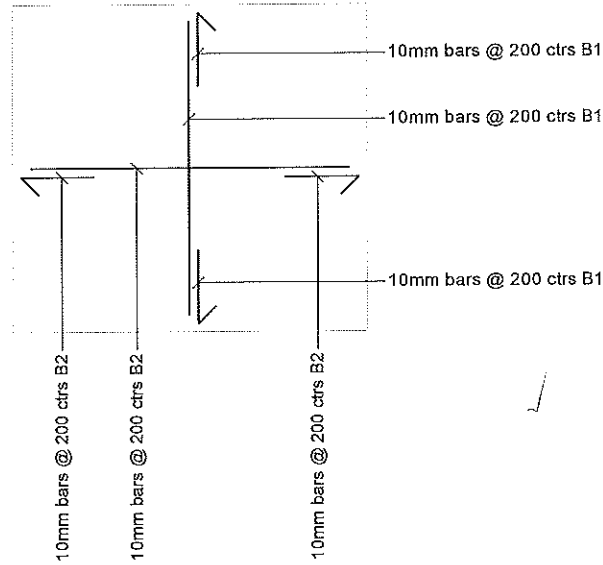
The following sketch is indicative only. Note that additional reinforcement may be required in accordance with clauses 9.2.1.2, 9.2.1.4 and 9.2.1.5 of EN 1992-1-1:2004 to meet detailing rules.



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Calculation Sheet

Project/Reference No: UR6306

Sheet No: 41

Project and Description: THE LODGE CROSS HOWELL
NEW DWELLING

Office: WREXHAM

Designed: IW

Checked:

FOUNDATION LOADING

CONSIDER LIVELOADING OF EXTERNAL + INTERNAL WALLS

LOADING

SIDE WALL

			DL	LL
ROOF + CEILING	$D.L = 1.18 \times 6.40 \text{ m} / 2$	$=$	3.78	5.10
	$I.L = 1.0 \times 6.40 \text{ m} / 2$	$=$	3.20	4.80
FIRST FLOOR	$D.L = 1.42 \times 4.60 \text{ m} / 2$	$=$	3.27	4.42
	$I.L = 1.5 \times 4.60 \text{ m} / 2$	$=$	3.45	5.18
GROUND FLOOR	$D.L = 5.5 \times 4.60 \text{ m} / 2$	$=$	12.66	17.09
	$I.L = 1.5 \times 4.60 \text{ m} / 2$	$=$	3.45	5.18
WALL	$D.L = 3.95 \times 5.3 \text{ m}$	$=$	20.94	28.27
			50.75	70.04
GARAGE (WITH FLOOR)				
FIRST FLOOR	$D.L = 1.42 \times 3.66 \text{ m} / 2$	$=$	2.60	3.51
	$I.L = 1.5 \times 3.66 \text{ m} / 2$	$=$	2.75	4.13
GROUND FLOOR	$D.L = 5.5 \times 3.66 \text{ m} / 2$	$=$	10.07	13.60
	$I.L = 1.5 \times 3.66 \text{ m} / 2$	$=$	2.75	4.13

Calculation Sheet

Project/Reference No: UR6306

Sheet No: 42

Project and Description: THE LODGE CROES HOWEL
NEW DWELLING

Office: WDEXHAM

Designed: IW

Checked:

GABLE LOAD (CONT.)			kN/m ²	
			SLS	ULS
WALL	D.L = $3.95 \times 5.3m$	=	20.94	28.27
INTERVAL - MAX LOAD			39.11	53.64
FIRST FLOOR	D.L = $1.42 \times \left(\frac{3.35+3.66}{2} \right)$	=	4.98	6.72
	I.L = $1.5 \times \left(\frac{3.35+3.66}{2} \right)$	=	5.26	7.89
GROUND FLOOR	D.L = $5.5 \times \left(\frac{3.35+3.66}{2} \right)$	=	19.28	26.03
	I.L = $1.5 \times \left(\frac{3.35+3.66}{2} \right)$	=	5.26	7.89
WALL	D.L = $2.0 \times 2.978m$	=	5.96	8.05
			40.74	56.58
DESIGN FOUNDATIONS BASED ON MAX LOAD - SIDEWALL				
D.L = $3.78 + 3.27 + 12.66 + 20.94 = 40.7 \text{ kN/m}$				
I.L = $3.2 + 3.45 + 3.45 = 10.1 \text{ kN/m}$				
FOR DESIGN SEE PG 43				
600W x 300D UNREINFORCED				
STRIP FOUNDATIONS OK				



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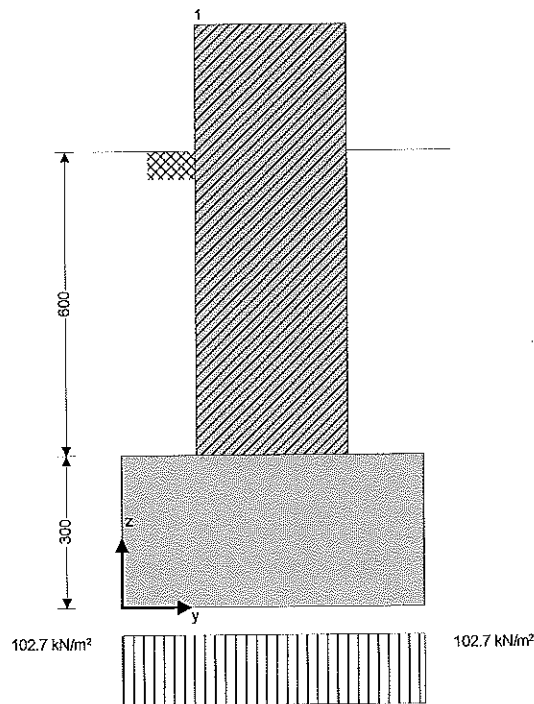
FOUNDATION ANALYSIS (EN1997-1:2004)

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

TEDDS calculation version 3.2.08

Strip foundation details - considering a one metre strip

Length of foundation	$L_x = 1000 \text{ mm}$
Width of foundation	$L_y = 600 \text{ mm}$
Foundation area	$A = L_x \times L_y = 0.600 \text{ m}^2$
Depth of foundation	$h = 300 \text{ mm}$
Depth of soil over foundation	$h_{\text{soil}} = 600 \text{ mm}$
Level of water	$h_{\text{water}} = 0 \text{ mm}$
Density of water	$\gamma_{\text{water}} = 9.8 \text{ kN/m}^3$
Density of concrete	$\gamma_{\text{conc}} = 24.5 \text{ kN/m}^3$



Wall no.1 details

Width of wall	$l_{y1} = 300 \text{ mm}$
position in y-axis	$y_1 = 300 \text{ mm}$

Soil properties

Density of soil	$\gamma_{\text{soil}} = 18.0 \text{ kN/m}^3$
Characteristic cohesion	$c'_k = 0 \text{ kN/m}^2$
Characteristic effective shear resistance angle	$\phi'_k = 30 \text{ deg}$
Characteristic friction angle	$\delta_k = 20 \text{ deg}$

Foundation loads

Self weight	$F_{\text{swt}} = h \times \gamma_{\text{conc}} = 7.4 \text{ kN/m}^2$
Soil weight	$F_{\text{soil}} = h_{\text{soil}} \times \gamma_{\text{soil}} = 10.8 \text{ kN/m}^2$



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Wall no.1 loads per linear metre

Permanent load in z

$$F_{Gz1} = 40.7 \text{ kN}$$

Variable load in z

$$F_{Qz1} = 10.1 \text{ kN}$$

Bearing resistance (Section 6.5.2)

Forces on foundation per linear metre

Force in z-axis

$$F_{dz} = A \times (F_{swt} + F_{soil}) + F_{Gz1} + F_{Qz1} = 61.6 \text{ kN}$$

Moments on foundation per linear metre

Moment in y-axis

$$M_{dy} = A \times (F_{swt} + F_{soil}) \times L_y / 2 + F_{Gz1} \times y_1 + F_{Qz1} \times y_1 = 18.5 \text{ kNm}$$

Eccentricity of base reaction

Eccentricity of base reaction in y-axis

$$e_y = M_{dy} / F_{dz} - L_y / 2 = 0 \text{ mm}$$

Pad base pressures

$$q_1 = F_{dz} \times (1 - 6 \times e_y / L_y) / (L_y \times 1000 \text{ mm}) = 102.7 \text{ kN/m}^2$$

$$q_2 = F_{dz} \times (1 + 6 \times e_y / L_y) / (L_y \times 1000 \text{ mm}) = 102.7 \text{ kN/m}^2$$

Minimum base pressure

$$q_{\min} = \min(q_1, q_2) = 102.7 \text{ kN/m}^2$$

Maximum base pressure

$$q_{\max} = \max(q_1, q_2) = 102.7 \text{ kN/m}^2$$

Presumed bearing capacity

Presumed bearing capacity

$$P_{\text{bearing}} = 103.0 \text{ kN/m}^2$$

PASS - Presumed bearing capacity exceeds design base pressure

*NB 3% OVER ASSUMED AVAILABLE GBP
HOWEVER OK BY INSPECTION*

Partial factors on actions - Combination1

Permanent unfavourable action - Table A.3

$$\gamma_G = 1.35$$

Permanent favourable action - Table A.3

$$\gamma_{Gr} = 1.00$$

Variable unfavourable action - Table A.3

$$\gamma_Q = 1.50$$

Variable favourable action - Table A.3

$$\gamma_{Qr} = 0.00$$

Partial factors for spread foundations - Combination1

Bearing - Table A.5

$$\gamma_{R,v} = 1.00$$

Sliding - Table A.5

$$\gamma_{R,h} = 1.00$$

Forces on foundation per linear metre

Force in z-axis

$$F_{dz} = \gamma_G \times (A \times (F_{swt} + F_{soil}) + F_{Gz1}) + \gamma_Q \times F_{Qz1} = 84.7 \text{ kN}$$

Moments on foundation per linear metre

Moment in y-axis

$$M_{dy} = \gamma_G \times (A \times (F_{swt} + F_{soil}) \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_Q \times F_{Qz1} \times y_1 = 25.4 \text{ kNm}$$

Eccentricity of base reaction

Eccentricity of base reaction in y-axis

$$e_y = M_{dy} / F_{dz} - L_y / 2 = 0 \text{ mm}$$

Effective area of base per linear metre

Effective width

$$L'_y = L_y - 2 \times e_y = 600 \text{ mm}$$

Effective length

$$L'_x = 1000 \text{ mm}$$

Effective area

$$A' = L'_x \times L'_y = 0.600 \text{ m}^2$$

Pad base pressure

Design base pressure

$$f_{dz} = F_{dz} / A' = 141.2 \text{ kN/m}^2$$

Partial factors on actions - Combination2

Permanent unfavourable action - Table A.3

$$\gamma_G = 1.00$$

Permanent favourable action - Table A.3

$$\gamma_{Gr} = 1.00$$

Variable unfavourable action - Table A.3

$$\gamma_Q = 1.30$$



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Variable favourable action - Table A.3

$$\gamma_{Qr} = 0.00$$

Partial factors for spread foundations - Combination2

Bearing - Table A.5

$$\gamma_{R,v} = 1.00$$

Sliding - Table A.5

$$\gamma_{R,h} = 1.00$$

Forces on foundation per linear metre

Force in z-axis

$$F_{dz} = \gamma_G \times (A \times (F_{swt} + F_{soil}) + F_{Gz1}) + \gamma_Q \times F_{Qz1} = 64.7 \text{ kN}$$

Moments on foundation per linear metre

Moment in y-axis

$$M_{dy} = \gamma_G \times (A \times (F_{swt} + F_{soil}) \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_Q \times F_{Qz1} \times y_1 = 19.4 \text{ kNm}$$

Eccentricity of base reaction

Eccentricity of base reaction in y-axis

$$e_y = M_{dy} / F_{dz} - L_y / 2 = 0 \text{ mm}$$

Effective area of base per linear metre

Effective width

$$L'_y = L_y - 2 \times e_y = 600 \text{ mm}$$

Effective length

$$L'_x = 1000 \text{ mm}$$

Effective area

$$A' = L'_x \times L'_y = 0.600 \text{ m}^2$$

Pad base pressure

Design base pressure

$$f_{dz} = F_{dz} / A' = 107.8 \text{ kN/m}^2$$

FOUNDATION DESIGN (EN1992-1-1:2004)

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

TEDDS calculation version 3.2.08

Concrete details (Table 3.1 - Strength and deformation characteristics for concrete)

Concrete strength class

$$C28/35$$

Characteristic compressive cylinder strength

$$f_{ck} = 28 \text{ N/mm}^2$$

Characteristic compressive cube strength

$$f_{ck,cube} = 35 \text{ N/mm}^2$$

Mean value of compressive cylinder strength

$$f_{cm} = f_{ck} + 8 \text{ N/mm}^2 = 36 \text{ N/mm}^2$$

Mean value of axial tensile strength

$$f_{ctm} = 0.3 \text{ N/mm}^2 \times (f_{ck} / 1 \text{ N/mm}^2)^{2/3} = 2.8 \text{ N/mm}^2$$

5% fractile of axial tensile strength

$$f_{ctk,0.05} = 0.7 \times f_{ctm} = 1.9 \text{ N/mm}^2$$

Secant modulus of elasticity of concrete

$$E_{cm} = 22 \text{ kN/mm}^2 \times [f_{cm} / 10 \text{ N/mm}^2]^{0.3} = 32308 \text{ N/mm}^2$$

Partial factor for concrete (Table 2.1N)

$$\gamma_C = 1.50$$

Compressive strength coefficient (cl.3.1.6(1))

$$\alpha_{cc} = 0.85$$

Design compressive concrete strength (exp.3.15)

$$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = 15.9 \text{ N/mm}^2$$

Tens. strength coeff. for plain concrete (cl.12.3.1(1))

$$\alpha_{ct,pl} = 0.80$$

Des. tens. strength for plain concrete (exp.12.1)

$$f_{ctd,pl} = \alpha_{ct,pl} \times f_{ctk,0.05} / \gamma_C = 1.0 \text{ N/mm}^2$$

Maximum aggregate size

$$h_{agg} = 20 \text{ mm}$$

Reinforcement details

Characteristic yield strength of reinforcement

$$f_{yk} = 500 \text{ N/mm}^2$$

Modulus of elasticity of reinforcement

$$E_s = 210000 \text{ N/mm}^2$$

Partial factor for reinforcing steel (Table 2.1N)

$$\gamma_S = 1.15$$

Design yield strength of reinforcement

$$f_{yd} = f_{yk} / \gamma_S = 435 \text{ N/mm}^2$$

Nominal cover to reinforcement

$$c_{nom} = 50 \text{ mm}$$

Strip and pad footings (Section 12.9.3)

Design base pressure

$$f_{dz} = 141.2 \text{ kN/m}^2$$

Projection from wall face

$$a = 150 \text{ mm}$$

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Project THE LODGE CROES HOWELL				Job no. VR6306	
Calcs for FOUNDATIONS				Start page no./Revision 46	
Calcs by IW	Calcs date 30/06/2016	Checked by	Checked date	Approved by	Approved date

Max.projection from wall face - (exp.12.13)

$$a_{\max} = 0.85 \times h / \sqrt{[3 \times f_{dz} / f_{ctd,pl}]} = 398 \text{ mm}$$

PASS - Projection from the wall face doesn't exceed permissible limit for plain concrete

Calculation Sheet

Project/Reference No: VR6306

Sheet No: 47

Project and Description: THE LODGECKOES HOUSE
NEW DWELLING

Office: WREXHAM

Designed: IN

Checked:

GARAGE PIER CHECK

AS SPECIFIED IN BUILDING REGULATION CONDITIONAL APPROVAL
ISOLATED CENTRAL PIER TO GARAGE DOOR REQUIRES CALCULATION

PIER WILL SUPPORT VERTICAL LOAD FROM DOOR LINTELS + WALL
ABOVE, PLUS LATERAL WIND LOAD

LOAD FROM LINTELS: 7.3KN EACH (PG 48)

SUPPORTING 215mm BRICK = $0.215 \times 20 = 4.3 \text{ KN/m}^2$

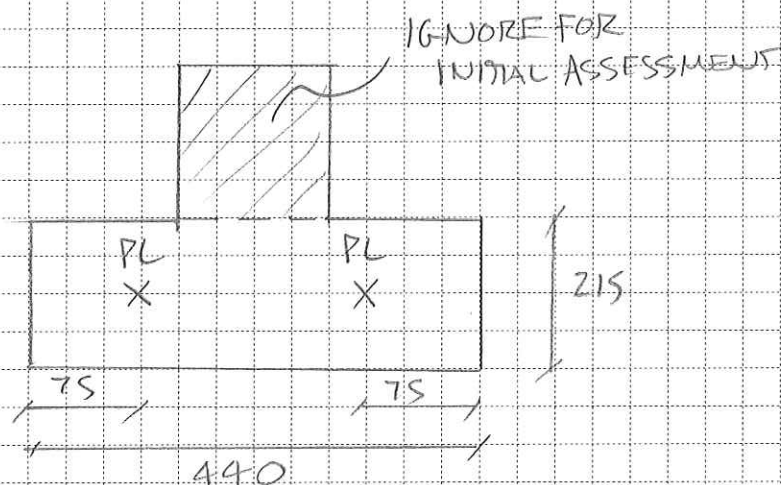
MIN $0.45\text{m} \times 4.3 = 1.935 \text{ KN/m}$

MAX $1.75\text{m} \times 4.3 = 7.525 \text{ KN/m}$

FROM ANALYSIS (PG 48) DL REACTION / LINTEL = 7.3KN

WIND LOAD ON PIER, SAY $1 \text{ KN/m}^2 \times (0.44 + 2.59\text{m}) = 3.03 \text{ KN/m}$

CONSIDER THE FOLLOWING:



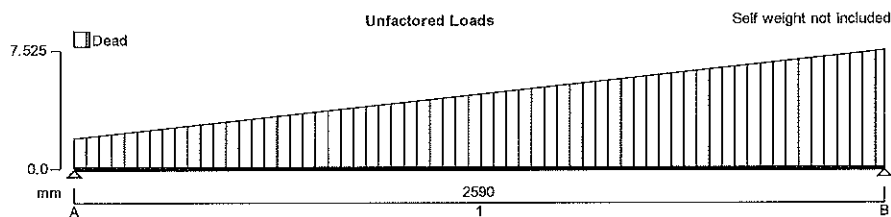
SEE PG 49

ISOLATED BRICK PIER IS SUITABLE FOR
ITS INTENDED APPLICATION



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Project				Job no.	
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GARAGE LINTEL ANALYSIS				48	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
IW	30/06/2016				



CONTINUOUS BEAM ANALYSIS - INPUT

BEAM DETAILS

Number of spans = 1

Material Properties:

Modulus of elasticity = 205 kN/mm²

Material density = 7860 kg/m³

Support Conditions:

Support A Vertically "Restrained"

Rotationally "Free"

Support B Vertically "Restrained"

Rotationally "Free"

Span Definitions:

Span 1 Length = 2590 mm

Cross-sectional area = 1000 mm²

Moment of inertia = 1.00×10⁶ mm⁴

LOADING DETAILS

Beam Loads:

Load 1 VDL Dead load 1.9 kN/m to 7.5 kN/m

LOAD COMBINATIONS

Load combination 1

Span 1 1.35×Dead

CONTINUOUS BEAM ANALYSIS - RESULTS

Unfactored support reactions

	Dead (kN)							
Support A	-4.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Support B	-7.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Support Reactions - Combination Summary

Support A	Max react = -6.6 kN	Min react = -6.6 kN	Max mom = 0.0 kNm	Min mom = 0.0 kNm
Support B	Max react = -9.9 kN	Min react = -9.9 kN	Max mom = 0.0 kNm	Min mom = 0.0 kNm

Beam Max/Min results - Combination Summary

Maximum shear = 6.6 kN	Minimum shear F_{min} = -9.9 kN
Maximum moment = 5.4 kNm	Minimum moment = 0.0 kNm
Maximum deflection = 18.3 mm	Minimum deflection = 0.0 mm



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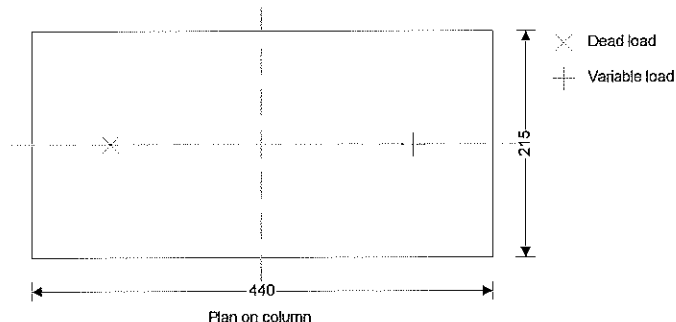
MASONRY COLUMN DESIGN

In accordance with EN1996-1-1:2005 incorporating corrigenda February 2006 and July 2009 and the UK national annex

Tedds calculation version 1.0.03

Geometry

Width of column $b = 440$ mm Thickness of column $t = 215$ mm
Height of column $h = 2100$ mm



Loading

Vertical dead load $G_k = 7.3$ kN Ecc dead load in x-direction $e_{Gx} = -145$ mm
Ecc dead load in y-direction $e_{Gy} = 0$ mm Vertical live load $Q_k = 7.3$ kN
Ecc variable load x-direction $e_{Qx} = 145$ mm Ecc variable load y-direction $e_{Qy} = 0$ mm
Characteristic wind loading $W_k = 1.0$ kN/m²

Masonry details

Masonry type Clay with water absorption less than 7% - Group 1
Normalised mean comp. str. $f_b = 20$ N/mm² Density of masonry $\gamma = 20$ kN/m³
Mortar type M6 - General purpose mortar
Comp. strength of mortar $f_m = 6$ N/mm² Compressive strength factor $K = 0.50$
Char. comp. strength masonry $f_k = 6.968$ N/mm² Char. flex. str. parallel to bed $f_{kx1} = 0.5$ N/mm²

Partial factors for material strength

Manufacturing control cat Category II Execution control class Class 2
Partial factor - comp flexure $\gamma_{Mc} = 3.00$

Slenderness ratio

Slenderness ratio minor axis $\lambda_t = 9.77$ Slenderness ratio major axis $\lambda_b = 4.77$
Maximum slenderness $\lambda = 9.77$

PASS - Slenderness ratio is less than 27

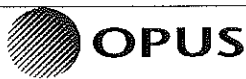
Load combinations derived from Eq 6.10 vertical loading (utilisation)

Combination 1 $1.35 \times \text{perm unfav} + 1 \times \text{perm fav} + 1.5 \times \text{variable} + 1.5 \times 0.5 \times \text{wind}$ (0.132)
Combination 2 $1.35 \times \text{perm unfav} + 1 \times \text{perm fav} + 1.5 \times 0.7 \times \text{variable} + 1.5 \times \text{wind}$ (0.129)

The following output relates to combination 1

Reduction factor for slenderness and eccentricity about the major axis - Section 6.1.2.2

Design moment at top of wall	$M_{tdb} = 0.2$ kNm	Design load at top of wall	$N_{tdb} = 20.8$ kN
Eccentricity at top of wall	$e_{tb} = 22$ mm	Reduction factor at top of wall	$\Phi_{tb} = 0.9$
Ratio of top and middle mnts	$\alpha_{mdb} = 1.0$	Design moment at mid wall	$M_{mdb} = 0.2$ kNm
Design load at mid wall	$N_{mdb} = 23.5$ kN	Eccentricity at mid wall	$e_{mkb} = 22$ mm
Reduction factor at mid wall	$\Phi_{mb} = 0.892$	Reduction factor	$\Phi_b = 0.892$



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Reduction factor for slenderness and eccentricity about the minor axis - Section 6.1.2.2

Design moment at top of wall	$M_{idt} = 0.0 \text{ kNm}$	Design load at top of wall	$N_{idt} = 20.8 \text{ kN}$
Eccentricity at top of wall	$e_{it} = 13.4 \text{ mm}$	Reduction factor at top of wall	$\Phi_{it} = 0.875$
Ratio of top and middle mnts	$\alpha_{mdt} = 1.0$	Design moment at mid wall	$M_{mdt} = 0.0 \text{ kNm}$
Design load at mid wall	$N_{mdt} = 23.5 \text{ kN}$	Eccentricity at mid wall	$e_{mkt} = 12.4 \text{ mm}$
Reduction factor at mid wall	$\Phi_{mt} = 0.826$	Reduction factor	$\Phi_t = 0.826$

Columns subjected to mainly vertical loading - Section 6.1.2

Design value vertical load	$N_{Ed} = 23.487 \text{ kN}$	Vertical resistance of column	$N_{Rd} = 178.491 \text{ kN}$
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PASS - Design vertical resistance exceeds applied design vertical load

Load combinations derived from Eq 6.10 lateral loading (utilisation)

Combination 1	$1.35 \times \text{perm unfav} + 1 \times \text{perm fav} + 1.5 \times \text{variable} + 1.5 \times 0.5 \times \text{wind} (0.203)$
Combination 2	$1.35 \times \text{perm unfav} + 1 \times \text{perm fav} + 1.5 \times 0.7 \times \text{variable} + 1.5 \times \text{wind} (0.405)$

The following output relates to combination 2

Reduction factor for slenderness and eccentricity about the major axis - Section 6.1.2.2

Design moment at top of wall	$M_{idb} = 0.3 \text{ kNm}$	Design load at top of wall	$N_{idb} = 17.5 \text{ kN}$
Eccentricity at top of wall	$e_{ib} = 32.9 \text{ mm}$	Reduction factor at top of wall	$\Phi_{ib} = 0.85$
Ratio of top and middle mnts	$\alpha_{mdb} = 1.0$	Design moment at mid wall	$M_{mdb} = 0.3 \text{ kNm}$
Design load at mid wall	$N_{mdb} = 20.2 \text{ kN}$	Eccentricity at mid wall	$e_{mkb} = 29.2 \text{ mm}$
Reduction factor at mid wall	$\Phi_{mb} = 0.859$	Reduction factor	$\Phi_b = 0.85$

Reduction factor for slenderness and eccentricity about the minor axis - Section 6.1.2.2

Design moment at top of wall	$M_{idt} = 0.0 \text{ kNm}$	Design load at top of wall	$N_{idt} = 17.5 \text{ kN}$
Eccentricity at top of wall	$e_{it} = 25.4 \text{ mm}$	Reduction factor at top of wall	$\Phi_{it} = 0.763$
Ratio of top and middle mnts	$\alpha_{mdt} = 1.0$	Design moment at mid wall	$M_{mdt} = 0.0 \text{ kNm}$
Design load at mid wall	$N_{mdt} = 20.2 \text{ kN}$	Eccentricity at mid wall	$e_{mkt} = 22.7 \text{ mm}$
Reduction factor at mid wall	$\Phi_{mt} = 0.727$	Reduction factor	$\Phi_t = 0.727$

Columns subjected to mainly vertical loading - Section 6.1.2

Design value vertical load	$N_{Ed} = 20.202 \text{ kN}$	Vertical resistance of column	$N_{Rd} = 157.126 \text{ kN}$
----------------------------	------------------------------	-------------------------------	-------------------------------

PASS - Design vertical resistance exceeds applied design vertical load

Flexural strength of masonry

Self weight at middle	$S_{wt} = 1.987 \text{ kN}$	Design vertical comp stress	$\sigma_d = 0.098 \text{ N/mm}^2$
Des flex str parallel to bed	$f_{xd1} = 0.167 \text{ N/mm}^2$	App flex str parallel to bed	$f_{xd1,app} = 0.265 \text{ N/mm}^2$

Column subject to lateral loading about the major axis - Section 6.3

Elastic section modulus	$Z_b = 6937333 \text{ mm}^3$	Mnt of resist parallel to bed	$M_{Rd1b} = 1.837 \text{ kNm}$
Bending moment coefficient	$\alpha = 0.125$	Design moment in column	$M_{Edb} = 0.178 \text{ kNm}$

PASS - Moment resistance greater than design moment in column

Column subject to lateral loading about the minor axis - Section 6.3

Elastic section modulus	$Z_t = 3389833 \text{ mm}^3$	Mnt of resist parallel to bed	$M_{Rdt} = 0.898 \text{ kNm}$
Bending moment coefficient	$\alpha = 0.125$	Design moment in column	$M_{Edt} = 0.364 \text{ kNm}$

PASS - Moment resistance greater than design moment in column

✓