

REPORT ON  
STRUCTURAL DESIGN OF  
STEEL FOOT BRIDGE ACROSS  
TALADANDA CANAL  
FOR TAKING CABLES

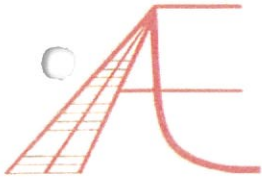
Client:

A.P Associates  
Janta Market,  
Rajouri Garden, New Delhi

Consultant:

Architectural, Construction & Engineering Consultancy  
A/10,HIG, Housing Board Colony,  
Barmunda, Bhubaneswar,  
Odisha - 751003





## **Report on Structural design of Steel Foot Bridge across Taladanda Canal for taking cables**

### **Introduction:**

A.P.Associates, JantaMarket, RajouriGarden, New Delhi has entrusted our firm for providing consultancy services for the GAD, design, drawing and BoQ of Steel Foot Bridge for taking cables over Taladanda Canal. The details of the consultancy services are given below.

### **Structural Design :**

The structural analysis and design has been done by limit state method using STAAD-Pro software. Rectangular Hollow Sections (RHS) and Square Hollow Sections (SHS) have been used in the steel foot bridge. The foot bridge is resting on Hinge Supports on both the ends and of length 53 m.

### **Design criteria:**

#### **I. Seismic parameters:**

Seismic zone- III as per IS 1893(Part-I)-2016

Soil type for seismic acceleration = Soft soil

Importance factor = 1.0

Reduction factor = 5

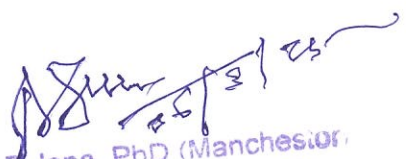
#### **II. Wind parameters:**

Wind velocity= 300km/hour (83.33 m/sec)

Terrain category = 2

Risk Coefficient (k1) = 1.00

Importance factor (k4) = 1.08

  
Prof. B. Jena, PhD (Manchester)  
Former Prof. NIT, Rourkela,  
Sr. Consultant (Civil)  
PRDC, Bhubaneswar

III. Load Combinations:

a) Limit states of collapse

1.5 DL + 1.5 LL

1.5 DL +/- 1.5 EL

1.5 DL +/- 1.5 WL

b) Limit states of serviceability

1.0 DL + 1.0 LL

1.0 DL +/- 1.0 EL

1.0 DL +/- 1.0 WL

IV. Environmental exposure condition = Moderate

V. Materials taken;

Grade of concrete = M30

Grade of steel = Fe 500

**Codes followed for structural design :**

IS 456:2000

IS 875: Part 2 -1987

IS 875: Part 3 -1987

IS 1893(Part-1) – 2016

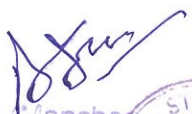
IS 13920 -2016

**Foundation design:**


Pile foundation in group having six number of piles of 600mm diameter and 10.5m depth below cutoff level with Pile cap has been provided at both ends as the SBC of soil is very low.

**Conclusion:**

The design report containing the calculation of wind load, Pile and Pile cap Design and design of hinge supports along with the structural drawings, prepared by Architectural, Construction and Engineering Consultancy, Bhubaneswar are submitted for necessary vetting/approval.

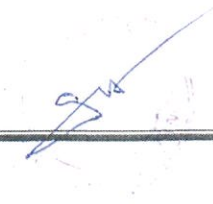
  
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Bijay Behera  
Architectural, Construction  
& Engineering Consultancy



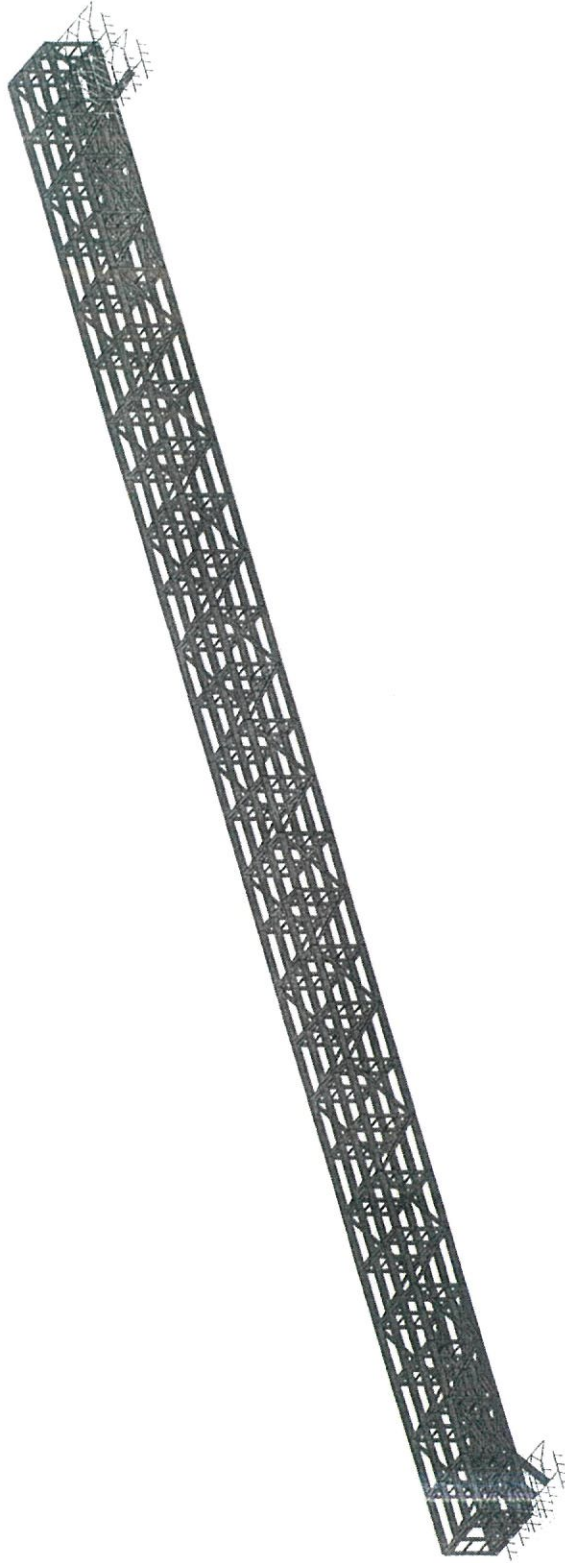
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Client		File taladanda Cable Bridge 2		Date/Time	13-Jul-2024 17:40
ABC CONSTRUCTIONS					

Software licensed to  
Job Title 30 m HIGH EXHAUST TOWER



31/07

STAAD SPACE FRAME

START JOB INFORMATION

JOB NAME 30 m HIGH EXHAUST TOWER

JOB CLIENT ABC CONSTRUCTIONS

JOB NO 1

JOB COMMENT LATTICE TOWER

ENGINEER NAME ACE CONSULTANCY

ENGINEER DATE 10.10.2016

END JOB INFORMATION

INPUT WIDTH 79

UNIT METER KN

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*Bijay*

*[Handwritten signature]*

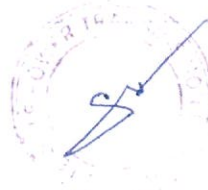


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\*SUPPORTS  
\*1 TO 3 FIXED  
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POISSON 0.17  
DENSITY 23.5615  
ALPHA 5.5e-006  
DAMP 0.05  
ISOTROPIC STEEL  
E 1.99947e+008  
POISSON 0.3  
DENSITY 76.8191  
ALPHA 6.5e-006  
DAMP 0.03  
END DEFINE MATERIAL  
MEMBER PROPERTY INDIAN  
1 TO 54 213 TO 268 275 TO 277 280 282 284 297 TO 300 307 TO 309 311 313 314 -  
367 TO 370 377 TO 379 381 383 384 393 TO 396 403 TO 405 407 409 410 -  
419 TO 422 429 TO 431 433 435 436 445 TO 448 455 TO 457 459 461 462 -  
471 TO 474 481 TO 483 485 487 488 497 TO 500 507 TO 509 511 513 514 -  
523 TO 526 533 TO 535 537 539 540 549 TO 552 559 TO 561 563 565 566 -  
575 TO 578 585 TO 587 589 591 592 601 TO 604 611 TO 613 615 617 618 -  
627 TO 630 637 TO 639 641 643 644 653 TO 656 663 TO 665 667 669 670 -  
679 TO 682 689 TO 691 693 695 696 705 TO 708 715 TO 717 719 721 722 -  
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835 TO 838 845 TO 847 849 851 852 861 TO 864 871 TO 873 875 877 878 -  
887 TO 890 897 TO 899 901 903 904 913 TO 916 923 TO 925 927 929 930 -  
939 TO 942 949 TO 951 953 955 956 965 TO 968 975 TO 977 979 981 982 -  
991 TO 994 1001 TO 1003 1005 1007 1008 1077 1080 1139 1142 1145 1148 1153 -  
1156 1179 1182 TABLE ST TUBE TH 0.006 WT 0.1 DT 0.1  
1183 TO 1194 1251 TO 1286 1291 TO 1294 1311 TO 1318 1355 1357 TO 1374 -  
1375 TABLE ST TUBE TH 0.006 WT 0.18 DT 0.18  
\*1183 TO 1194 1251 TO 1326 TABLE ST TUBE TH 0.006 WT 0.1 DT 0.1  
107 TO 133 186 TO 212 271 274 278 283 287 288 291 292 301 TO 304 323 TO 366 -  
371 TO 374 397 TO 400 423 TO 426 449 TO 452 475 TO 478 501 TO 504 -  
527 TO 530 553 TO 556 579 TO 582 605 TO 608 631 TO 634 657 TO 660 -  
683 TO 686 709 TO 712 735 TO 738 761 TO 764 787 TO 790 813 TO 816 -  
839 TO 842 865 TO 868 891 TO 894 917 TO 920 943 TO 946 969 TO 972 -  
995 TO 998 1017 TO 1076 TABLE ST TUBE TH 0.006 WT 0.18 DT 0.18  
269 270 279 281 285 286 289 290 293 TO 296 305 306 310 312 315 TO 322 375 -



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376 380 382 385 TO 392 401 402 406 408 411 TO 418 427 428 432 434 -  
437 TO 444 453 454 458 460 463 TO 470 479 480 484 486 489 TO 496 505 506 -  
510 512 515 TO 522 531 532 536 538 541 TO 548 557 558 562 564 567 TO 574 -  
583 584 588 590 593 TO 600 609 610 614 616 619 TO 626 635 636 640 642 645 -  
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926 928 931 TO 938 947 948 952 954 957 TO 964 973 974 978 980 983 TO 990 -  
999 1000 1004 1006 1009 TO 1016 TABLE ST TUBE TH 0.0045 WT 0.05 DT 0.05  
55 TO 106 1081 TO 1132 1356 1381 TABLE ST TUBE TH 0.008 WT 0.12 DT 0.24  
134 TO 185 1199 TO 1250 TABLE ST TUBE TH 0.008 WT 0.12 DT 0.24  
\*1347 TO 1354 TABLE ST TUBE TH 0.008 WT 0.15 DT 0.3  
1347 TO 1354 TABLE ST TUBE TH 0.008 WT 0.12 DT 0.24  
CONSTANTS  
MATERIAL STEEL ALL  
\*START USER TABLE  
\*TABLE 1  
\*TABLE FR ISMC200  
\*TABLE FR ISMC150  
\*TABLE FR ISMC75  
\*END  
\*BUILT-UP SECTION S1 2ISM200  
\*1 TO 20 481 484 486 -  
\*489 PRIS AX 56420 IY 3.6386e+007 IZ 2.99544e+009 YD 200 ZD 150  
\*BUILT-UP SECTION S1 2ISM150  
\*21 TO 60 PRIS AX 41760 IY 1.5588e+007 IZ 2.21404e+009 YD 150 ZD 150  
\*BUILT-UP SECTION S1 2ISM75  
\*61 TO 80 PRIS AX 17340 IY 1.52e+006 IZ 2.59817e+008 YD 75 ZD 80  
\*DEFINE WIND LOAD  
\*TYPE 1  
\*INT 0 1.5 HEIG 1 11  
\*EXP 1 YR 1 20  
SUPPORTS  
1 27 28 54 113 117 681 685 803 TO 810 FIXED BUT MX  
\*UNIT MMS KN  
DEFINE 1893 LOAD  
ZONE 0.16 RF 5 I 1 SS 3 ST 2 DM 0.02 DT 8  
SELFWEIGHT 1  
JOINT WEIGHT  
123 124 WEIGHT 0.352  
121 122 127 TO 129 689 690 693 WEIGHT 0.26  
125 126 130 TO 132 691 692 694 TO 696 WEIGHT 0.111  
153 154 157 158 161 162 165 166 169 170 173 174 177 178 181 182 185 186 189 -  
190 193 194 697 698 701 702 705 706 709 710 713 714 717 718 721 722 725 726 -  
729 730 733 734 737 738 741 742 745 746 749 750 WEIGHT 0.704  
145 146 149 155 156 159 160 163 164 167 168 171 172 175 176 179 180 183 184 -  
187 188 191 192 195 196 209 210 213 229 230 233 249 250 253 269 270 273 289 -  
290 293 309 310 313 329 330 333 349 350 353 369 370 373 389 390 393 409 410 -  
413 429 430 433 449 450 453 469 470 473 489 490 493 509 510 513 529 530 533 -  
549 550 553 569 570 573 589 590 593 609 610 613 629 630 633 649 650 653 669 -  
670 673 699 700 703 704 707 708 711 712 715 716 719 720 723 724 727 728 731 -  
732 735 736 739 740 743 744 747 748 751 752 WEIGHT 0.52  
147 148 150 TO 152 211 212 214 TO 216 231 232 234 TO 236 251 252 254 TO 256 -  
271 272 274 TO 276 291 292 294 TO 296 311 312 314 TO 316 331 332 334 TO 336 -  
351 352 354 TO 356 371 372 374 TO 376 391 392 394 TO 396 411 412 414 TO 416 -  
431 432 434 TO 436 451 452 454 TO 456 471 472 474 TO 476 491 492 494 TO 496 -  
511 512 514 TO 516 531 532 534 TO 536 551 552 554 TO 556 571 572 574 TO 576 -  
591 592 594 TO 596 611 612 614 TO 616 631 632 634 TO 636 651 652 654 TO 656 -  
671 672 674 TO 676 WEIGHT 0.222  
FLOOR WEIGHT  
YRANGE 0 0.1 FLOAD 0.5 XRANGE -1 53 ZRANGE 0.8 1.8  
LOAD 1 SEISMIC ALONG +X  
1893 LOAD X 1  
LOAD 2 SEISMIC ALONG -X  
1893 LOAD X -1  
LOAD 3 SEISMIC ALONG +Z  
1893 LOAD Z 1  
LOAD 4 SEISMIC ALONG -Z  
1893 LOAD Z -1





LOAD 5 SELFWT & WALL LOAD

SELFWEIGHT Y -1

JOINT LOAD

123 124 FY -0.352

121 122 127 TO 129 689 690 693 FY -0.26

125 126 130 TO 132 691 692 694 TO 696 FY -0.111

153 154 157 158 161 162 165 166 169 170 173 174 177 178 181 182 185 186 189 -

190 193 194 697 698 701 702 705 706 709 710 713 714 717 718 721 722 725 726 -

729 730 733 734 737 738 741 742 745 746 749 750 FY -0.704

145 146 149 155 156 159 160 163 164 167 168 171 172 175 176 179 180 183 184 -

187 188 191 192 195 196 209 210 213 229 230 233 249 250 253 269 270 273 289 -

290 293 309 310 313 329 330 333 349 350 353 369 370 373 389 390 393 409 410 -

413 429 430 433 449 450 453 469 470 473 489 490 493 509 510 513 529 530 533 -

549 550 553 569 570 573 589 590 593 609 610 613 629 630 633 649 650 653 669 -

670 673 699 700 703 704 707 708 711 712 715 716 719 720 723 724 727 728 731 -

732 735 736 739 740 743 744 747 748 751 752 FY -0.52

147 148 150 TO 152 211 212 214 TO 216 231 232 234 TO 236 251 252 254 TO 256 -

271 272 274 TO 276 291 292 294 TO 296 311 312 314 TO 316 331 332 334 TO 336 -

351 352 354 TO 356 371 372 374 TO 376 391 392 394 TO 396 411 412 414 TO 416 -

431 432 434 TO 436 451 452 454 TO 456 471 472 474 TO 476 491 492 494 TO 496 -

511 512 514 TO 516 531 532 534 TO 536 551 552 554 TO 556 571 572 574 TO 576 -

591 592 594 TO 596 611 612 614 TO 616 631 632 634 TO 636 651 652 654 TO 656 -

671 672 674 TO 676 FY -0.222

LOAD 6 STAIR WATER TANK LIFT

LOAD 7 DEAD FLOOR LOAD

FLOOR LOAD

YRANGE 0 0.1 FLOAD -0.5 XRANGE -1 53 ZRANGE 0.8 1.8 GY

LOAD 8 LIVE LOAD

FLOOR LOAD

YRANGE 0 0.1 FLOAD -0.8 XRANGE -1 53 ZRANGE 0.8 1.8 GY

LOAD 9 EQLIVE FLOOR LOAD

FLOOR LOAD

YRANGE 0 0.1 FLOAD -0.2 XRANGE -1 53 ZRANGE 0.8 1.8 GY

\*WIND LOAD

LOAD 10 WIND LOAD IN X DIRECTION

LOAD 11 WIND LOAD IN -X DIRECTION

LOAD 12 WIND LOAD IN Z DIRECTION

MEMBER LOAD

28 TO 54 215 TO 236 267 268 299 300 369 370 395 396 421 422 447 448 473 474 -

499 500 525 526 551 552 577 578 603 604 629 630 655 656 681 682 707 708 733 -

734 759 760 785 786 811 812 837 838 863 864 889 890 915 916 941 942 967 968 -

993 994 1145 UNI GZ -0.627

81 TO 106 160 TO 185 UNI GZ -1.506

1 TO 27 241 TO 262 265 266 275 TO 277 280 282 284 297 298 307 TO 309 311 313 -

314 367 368 377 TO 379 381 383 384 393 394 403 TO 405 407 409 410 419 420 -

429 TO 431 433 435 436 445 446 455 TO 457 459 461 462 471 472 481 TO 483 -

485 487 488 497 498 507 TO 509 511 513 514 523 524 533 TO 535 537 539 540 -

549 550 559 TO 561 563 565 566 575 576 585 TO 587 589 591 592 601 602 611 -

612 TO 613 615 617 618 627 628 637 TO 639 641 643 644 653 654 663 TO 665 667 -

669 670 679 680 689 TO 691 693 695 696 705 706 715 TO 717 719 721 722 731 -

732 741 TO 743 745 747 748 757 758 767 TO 769 771 773 774 783 784 -

793 TO 795 797 799 800 809 810 819 TO 821 823 825 826 835 836 845 TO 847 -

849 851 852 861 862 871 TO 873 875 877 878 887 888 897 TO 899 901 903 904 -

913 914 923 TO 925 927 929 930 939 940 949 TO 951 953 955 956 965 966 975 -

976 TO 977 979 981 982 991 992 1001 TO 1003 1005 1007 1008 -

1148 UNI GZ -0.481

55 TO 80 134 TO 159 1081 TO 1132 1199 TO 1250 1356 1381 UNI GZ -1.155

213 214 237 238 1077 1139 1156 1182 UNI GZ -0.627

239 240 263 264 1080 1142 1153 1179 UNI GZ -0.481

1183 TO 1194 1251 TO 1286 1291 TO 1294 1311 TO 1318 UNI GZ -0.866

LOAD 13 WIND LOAD IN -Z DIRECTION

\*WITH 5% EXTRA FOR WEIGHT FOR GUSSETS & LADDER

\*LOAD FACTORS 1.05X1.5=1.575

LOAD COMB 14 (1.5DL + 1.5LL)

5 1.5 7 1.5 8 1.5

LOAD COMB 15 (1.5 DL + 1.5 WL)

5 1.5 7 1.5 10 1.5

LOAD COMB 16 (1.5 DL + 1.5 WL)

5 1.5 7 1.5 11 1.5

LOAD COMB 17 (1.5 DL + 1.5 WL)



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5 1.5 7 1.5 12 1.5  
LOAD COMB 18 (1.5 DL + 1.5 WL)  
5 1.5 13 1.575  
\*LOAD FACTORS 1.1X0.9=0.99  
LOAD COMB 19 (0.9 DL + 1.5 WL)  
5 0.99 10 1.5  
LOAD COMB 20 (0.9 DL + 1.5 WL)  
5 0.99 11 1.5  
LOAD COMB 21 (0.9 DL + 1.5 WL)  
5 0.99 12 1.5  
LOAD COMB 22 (0.9 DL + 1.5 WL)  
5 0.99 13 1.5  
LOAD COMB 23 (1.5 DL + 1.5 EL)  
1 1.5 5 1.5 9 1.5  
LOAD COMBINATION 24  
2 1.5 5 1.5 9 1.5  
LOAD COMBINATION 25  
3 1.5 5 1.5 9 1.5  
LOAD COMBINATION 26  
4 1.5 5 1.5 9 1.5  
LOAD COMB 27 (0.9 DL + 1.5 EL)  
1 1.5 5 0.99  
LOAD COMBINATION 28  
2 1.5 5 0.99  
LOAD COMBINATION 29  
3 1.5 5 0.99  
LOAD COMBINATION 30  
4 1.5 5 0.99  
\*WIND BLOWING FROM CORNER  
\*EACH COMPONENT 1.0xSIN 45=0.707  
\*1.2\*0.707=0.848  
\*1.2X1.1=1.32  
LOAD COMB 31 (1.2 DL + 0.848 WL IN X + 0.848 WL IN Z)  
5 1.32 10 0.848 12 0.848  
LOAD COMB 32 (1.2 DL + 0.848 WL IN Z + 0.848 WL IN -X)  
5 1.32 12 0.848 11 0.848  
LOAD COMB 33 (1.2 DL + 0.848 WL IN -X + 0.848 WL IN -Z)  
5 1.32 11 0.848 13 0.848  
LOAD COMB 34 (1.2 DL + 0.848 WL IN -Z + 0.848 WL IN X)  
5 1.32 13 0.848 10 0.848  
\*FOR SUPPORT REACTIONS  
LOAD COMB 35 (1.0 DL + 1.0 WL)  
5 1.0 7 1.0 10 1.0  
LOAD COMB 36 (1.0 DL + 1.0 WL)  
5 1.0 7 1.0 11 1.0  
LOAD COMB 37 (1.0 DL + 1.0 WL)  
5 1.0 7 1.0 12 1.0  
LOAD COMB 38 (1.0 DL + 1.0 WL)  
5 1.0 7 1.0 13 1.0  
LOAD COMB 39 (1.0 DL + 1.0 EL)  
1 1.0 5 1.0 7 1.0  
LOAD COMBINATION 40  
2 1.0 5 1.0 7 1.0  
LOAD COMBINATION 41  
3 1.0 5 1.0 7 1.0  
LOAD COMBINATION 42  
4 1.0 5 1.0 7 1.0  
\*WIND BLOWING FROM CORNER  
\*EACH COMPONENT 1.0\*SIN 45=0.707  
LOAD COMB 43 (1.1 DL + 0.707 WL IN X + 0.707 WL IN Z )  
5 1.1 10 0.707 12 0.707  
LOAD COMB 44 (1.1 DL + 0.707 WL IN Z + 0.707 WL IN -X)  
5 1.1 12 0.707 11 0.707  
LOAD COMB 45 (1.1 DL + 0.707 WL IN -X + 0.707 WL IN -Z)  
5 1.1 11 0.707 13 0.707  
LOAD COMB 46 (1.1 DL + 0.707 WL IN -Z + 0.707 WL IN X)  
5 1.1 13 0.707 10 0.707  
LOAD COMB 47 (1.0DL + 1.0LL)  
5 1.0 7 1.0 8 1.0  
PERFORM ANALYSIS





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LOAD LIST 35 TO 42 47

PRINT SECTION MAX DISPL NSECT 3 LIST ALL

LOAD LIST 14 TO 18 23 TO 26

PARAMETER 1

CODE IS800 LSD

KY 0.85 ALL

KZ 0.85 ALL

FYLD 310000 ALL

\*NSF 1 MEMB 1 84

CHECK CODE ALL

STEEL TAKE OFF LIST ALL

FINISH

MEMBER RELEASE

269 305 375 401 427 453 479 505 531 557 583 609 635 661 687 713 739 765 791 -

817 843 869 895 921 947 973 999 START MZ

293 319 389 415 441 467 493 519 545 571 597 623 649 675 701 727 753 779 805 -

831 857 883 909 935 961 987 1013 END MZ

270 306 376 402 428 454 480 506 532 558 584 610 636 662 688 714 740 766 792 -

818 844 870 896 922 948 974 1000 START MZ

296 322 392 418 444 470 496 522 548 574 600 626 652 678 704 730 756 782 808 -

834 860 886 912 938 964 990 1016 END MZ

290 318 388 414 440 466 492 518 544 570 596 622 648 674 700 726 752 778 804 -

830 856 882 908 934 960 986 1012 END MZ

286 316 386 412 438 464 490 516 542 568 594 620 646 672 698 724 750 776 802 -

828 854 880 906 932 958 984 1010 END MZ

279 310 380 406 432 458 484 510 536 562 588 614 640 666 692 718 744 770 796 -

822 848 874 900 926 952 978 1004 START MZ

281 312 382 408 434 460 486 512 538 564 590 616 642 668 694 720 746 772 798 -

824 850 876 902 928 954 980 1006 START MZ

CODE IS800 LSD

KY 0.85 ALL

KZ 0.85 ALL

FYLD 250000 ALL



*[Handwritten signature]*

CALCULATION FOR  
WIND PRESURE



Wind Pressure:

Basic wind speed from IS 875 (Part-3)- 1987

Terrain category	Vb	=	77.2 m/sec	=	277.92 Km/Hour
Type of building / structure	h	=	2		
Probability Factor		=	5.5		
Terrain, Height & Structure factor	K1	=	Permanent shed		
	K2	=	1.00 upto 10 m		
		=	1.00 10 to 15 m		
		=	1.07 15 to 20 m		
		=	1.00		
		=	1.08		
Topography Factor	K3	=	K1 x K2 x K3 x K4 x Vb		
	K4	=	1.00 x		
	Vz	=	83.376 m/sec	1.00 x	1.00 x 77
		=	1.00 x	upto 10 m	1.08 x 77
		=	87.5448 m/sec	1.05	1.08 x 77
		=	1.00 x	10 to 15 m	1.08 x 77
		=	89.21232 m/sec	1.07	1.08 x 77
		=		15 to 20 m	
		=			300.1536
Design Wind Pressure	Pz	=	0.6 Vz <sup>2</sup>		
		=	0.6 x	83.376 <sup>2</sup>	
		=	4170.9344 N/Sqm		
		=	4.171 KN/Sqm	upto 10 m	
		=	0.6 x	87.5448 <sup>2</sup>	
		=	4598.4552 N/Sqm		
		=	4.598 KN/Sqm	10 to 15 m	
		=	0.6 x	89.2123 <sup>2</sup>	
		=	4775.3028 N/Sqm		
		=	4.78 KN/Sqm	15 to 20 m	
Wind directionality factor	Pd	=	Kd Ka Kc Pz		
	Kd	=	1.00		
Area averaging factor	Ka	=	Tributary area in sqm		
		=	1.00 10		
		=	0.90 25		
		=	0.80 100		
Tributary area		=	36.413 x	1	

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Ka	=	36.41	-	0.90	-	0.80 x -11.41
	=	0.90		25	-	100
Kc Kd Ka Kc Pz Minimum upto 10m Pd	=	0.88				
	=	1.00				
	=	0.88	Pz			
	=	0.70	Pz			
	=	0.88			4.171	
	=	3.69				x kN/Sqm

2



  
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# SOLIDITY RATIO

Length	=	53 m	
Depth	=	1.75 m	
c/c of section	=	0.3 m	
		<u>2.05 m</u>	
Total area of the frame normal to wind direction	=	53 x	2.05
	=	108.65 Sqm	
Obstructed area	=	No	L
		2	53
		26	2.59
		27	1.57
			B
			0.24
			0.1
			0.1
			<u>25.44</u>
			6.734
			4.239
			<u>36.413 sqm</u>

Solidity ratio

$$= \frac{36.413}{108.65}$$

0.34

Force coefficients for single frames Ct

1.7 rom table 31

Solidity Ratio

0.3 =

0.4 =

0.34 =

pd

Force on frame

Width

Force/m

Front frame

0.627

1.506

1.129

kN/sqm

kN/sqm

Frame spacing ratio = C/c distance between frames, beams or girders

Least overall dimension of the frames, beam or girder measured in a direction normal to the direction of wind

0.88

1.75

0.503

Effective solidity ratio

Area of obstruction by front frame = 36.413 sqm

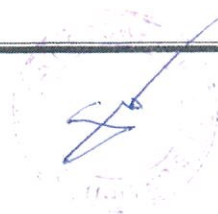
Length of exposed cable = 53 - 27 x 0.1



*[Signature]*  
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# DESIGN OF HINGE SUPPORT



### Design of Hinge for top support

Support reaction at joint

Load Comb.	Fx	Fy	Fz
DL + LL	-79.22	-105.57	17.33
Service load	-118.83	-158.355	25.995
Design Load	-1034.64	-98.16	173.92
DL+WL	-1551.96	-147.24	260.88
Service load			
Design Load			

### Design of Pin

P is the pull (Maximum of two)

$$P = V \quad 0^2 \quad + \quad 158.355^2$$

$$P = V \quad + \quad V \quad + \quad 147.24 \text{ }^2$$

Let us take

fyp	=	250	N/sqmm
fu	=	410	N/sqmm

## 1 Shear Capacity

As the pin is subjected to rotation

shear capacity

0.5 fyp A

where  $A$  is the cross sectional area of the pin

$f_{yp}$  is the yield strength of the pin

The pin will fail in double shear


Shear force acting on the pin

[illegible]

Dia. of pin

Let us provide pin dia.



  
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### Bearing Capacity

As the pin is subjected to rotation

$$\text{Bearing capacity} = \frac{0.8 f_y d t}{2}$$

where d is the diameter of the pin

$f_y$  is the lower yield strength of the pin & connected part

The pin is resting on two plates

Force on each plate	=	$\frac{158.355}{2}$ kN		
	=	79.1775	kN	
$f_y$	=	240	N/sqmm	
dt	=	79.1775	x	1000
	=	0.8	x	240
	=	412.3828125	sqmm	
Dia. of pin taken	=	50	mm	
	=	412.3828125		
	=	50		
	=	8.24765625	mm	
Let us provide thickness of supporting plate	=	16	mm	

### 3 Bending

As the pin is subjected to rotation

$$\text{Moment capacity} = \frac{1 f_y p Z}{2}$$

where  $f_y$  is the yield strength of the pin

Z is the section modulus of the pin

The pin is resting on two plates


$$\text{Moment on the pin} = \frac{PL}{4}$$

Let us assume

$$\text{Thickness of connecting plate} = 20 \text{ mm}$$

from shear consideration

Shear capacity	=	0.5	$f_y p A$	
Width of plate around pin	=	$\frac{158.355}{250}$	x	$\frac{1000}{20}$

  
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$$\begin{aligned}
 &= 15.8355 \text{ mm} \\
 &\text{Let us provide width of connecting plate around pin} \\
 &= 30 \text{ mm} \\
 &\text{from bearing consideration} \\
 &= 0.8 f_y d t \\
 &= 158.355 \text{ kN} \\
 &= 158.355 \times 250 \times 1000 \\
 &= 0.8 \times 158.355 \times 250 \times 50 \\
 &= 15.8355 \text{ mm}
 \end{aligned}$$

$$\begin{aligned}
 &\text{Let us provide} \\
 &\text{thickness of connecting plate} \\
 &= 20 \text{ mm} \\
 &\text{Recess in between} \\
 &= 6 \\
 &\text{Span} \\
 &= 42 \text{ mm} \\
 &= 158.355 \times 4 \\
 &= 0.042
 \end{aligned}$$

$$\begin{aligned}
 M &= 158.355 \times 4 \\
 &= 1.663 \text{ kNm} \\
 Z &= \frac{\pi d^3}{32} \\
 &= \frac{12271.8463 \text{ mm}^3}{32} \\
 d^3 &= \frac{1662727.500 \times 32}{\pi} \\
 &= 250 \times 67745.61296 \text{ mm}^3 \\
 &= 40.76558937 \text{ mm} \\
 d &= 50 \text{ mm} \\
 &\text{or say} \\
 &= 50 \text{ mm}
 \end{aligned}$$

Dia of pin provided


Design of triangular plate connected to base plate for holding the pin

From shear consideration

Resultant Load acting on the plate

$$\begin{aligned}
 &= \frac{158.355 \text{ kN}}{2} \\
 &= 79.1775 \text{ kN} \\
 &= 79.1775 \\
 &= \frac{79.1775 \times 1000}{0.5 \times 250 \times}
 \end{aligned}$$



  
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=  $t$  316.71 sqmm  
 = 16 mm  
 = Width 316.71  
 16

= 19.794375  
 = 50 mm

Let us provide min. width around pin  
 Width at bottom  
 Resultant Load acting on the plate

= 158.355 kN  
 = 2

= 79.1775 kN

= 0.5 fyp A  
 = A

= 79.1775

= 79.1775 x 1000

= 0.5 x 250

= 633.42 sqmm

= 16

= 633.42

=  $t$   
 = Width

= 39.58875

= 50 mm

= 30 mm

= >

Let us provide min. width around pin

Width of connecting plate to strut of frame

Maximum of following two

Resultant Force action on the plate

Member force from staad

Force acting on connecting plate

Tension Capacity of Plate

= 158.355 kN

= 0.000 kN

= 158.355 kN

= 0.900 fuAn

= 1.250

= 0.720 fuAn

= 158.355 kN

= 0.720 fu An  
 = w

= 158.355 x

= 410 x 1000

= 20

= 26.82164634

Min. width of connecting plate to be provided

= dia. of pin

= 50

= + twice of width around pin

= + (

= 2

= x

= 30 )

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### Shear Check

=	110	mm
or say	300	x 20 mm
Shear on plate from STAAD	5.97	kN
0.5 fyp A	5.97	
Shear capacity of plate	0.5	x 158.355 x 300 x 20
	475065	N
	475.065	kN
	>	5.97 kN

### Connection between connecting plate & tube

External width of pipe	120 mm
Thickness of pipe	8 mm
Let us provide a flange plate of size	350 x 12

Let us weld the circular plate to the pipe by providing fillet weld

Welding between circular plate & connecting plate

i) Length of weld by welding on both sides	2 x ( 300 + 20 )
	640 mm
Thickness of weld required	158.355 x 1000
	0.7 x 158 x 640
	2.237158115 mm
Let us provide weld thickness	8 mm

### Welding of supporting triangular plate with base plate

Required Length of connection of plate with base	( 2 x 156 mm ) + 50
	200 mm
Let us provide	50 mm
Edge distance of plate on both sides	200 + ( 2 x 50 )
Length of base plate	300 mm
	300 x 16
Let us provide	8 mm
Let us take size of fillet weld	2 x ( 632 mm )
Effective length of weld	300 + 16
	300 x 16

Force on the weld is maximum of the following two values

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$$\begin{aligned} &= V \quad 118.83^2 + 158.355^2 + 25.995^2 \\ &= V \quad 14120.57 + 675.74003 \\ &= V \quad 39872.61 \\ &= 199.6813 \text{ kN} \end{aligned}$$

$$\begin{aligned}
 &= V & 1551.96^2 & + & 147.24^2 & + & 260.88^2 \\
 &= V & 2408580 & + & 68058.374 & & \\
 &= V & 2498318 & & & & \\
 &= & 1580.607 \text{ kN} & & & &
 \end{aligned}$$

ii) PdW

[illegible]

Let us provide size of fillet weld

  
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### Design of Hinge for top support

Support reaction at joint

Load Comb.	Fx	Fy	Fz
DL + LL	332.96	162.94	-0.75
Design Load	499.44	244.41	-1.125
DL+WL	550.65	266.01	0.36
Design Load	825.975	399.015	0.54

### Design of Pin

P is the pull (Maximum of two)

i)	P	=	V	=	499.44 <sup>2</sup>	+	244.41 <sup>2</sup>
		=			556.036	kN	
ii)	P	=	V	=	825.975 <sup>2</sup>	+	399.015 <sup>2</sup>
		=			917.305	kN	
	P	=			917.305	kN	
Let us take	f <sub>yp</sub>	=			250	N/sqmm	
	f <sub>u</sub>	=			410	N/sqmm	

### 1 Shear Capacity

As the pin is subjected to rotation shear capacity

$$= 0.5 f_{yp} A$$

where A is the cross sectional area of the pin  
f<sub>yp</sub> is the yield strength of the pin

The pin will fail in double shear  
Shear force acting on the pin

=					917.305	kN	
=					$\frac{2}{2}$		
=					458.6522841	kN	
=	f <sub>yp</sub>				250	N/sqmm	
=	A				458.6522841	x	1000
					0.5	x	250
=					3669.218273	sqmm	
=					V (		3669.2183 )
					$\frac{\pi}{4} \times$		
=					68.3505216		
=					100	mm	

Dia. of pin

Let us provide pin dia.



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### Bearing Capacity

As the pin is subjected to rotation

$$\text{Bearing capacity} = 0.8 f_y d t$$

where d is the diameter of the pin

$f_y$  is the lower yield strength of the pin & connected part

The pin is resting on number of plates

Force on each plate

2	=	917.305	kN
		2	
	=	458.6522841	kN
	=	240 N/sqmm	
	=	458.6522841	x
		0.8	x
	=	2388.81398	sqmm
	=	100	mm
	=	2388.81398	
		100	
	=	23.8881398	mm
	=	25	mm

Dia. of pin taken

d

t

Let us provide thickness of supporting plate

### 3 Bending

As the pin is subjected to rotation

$$\text{Moment capacity} = 1 f_y Z$$

where  $f_y$  is the yield strength of the pin

Z is the section modulus of the pin

The pin is resting on two plates

$$PL/4$$

Moment on the pin

Let us assume

Thickness of connecting plate

$$50 \text{ mm}$$

from shear consideration

$$\text{Shear capacity} = 0.5 f_y A$$

Width of plate around pin

917.305	x	1000
250	x	50 x
		2

*18/12/20*  
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$$\begin{aligned}
 &= \text{Let us provide width of connecting plate} \\
 &= \text{around pin} \\
 &= \text{from bearing consideration} \\
 &= 0.8 f_y d t \\
 &= 36.692 \text{ mm} \\
 &= 60 \text{ mm} \\
 &= 917.305 \text{ kN} \\
 &= 917.305 \times \frac{1000}{250 \times 100} \\
 &= 0.8 \times 45.86522841 \text{ mm} \\
 &= 36.692 \text{ mm}
 \end{aligned}$$

$$\begin{aligned}
 &= \text{Let us provide} \\
 &= \text{thickness of connecting plate} \\
 &= \text{Recess in between} \\
 &= \text{Span} \\
 &= 50 \text{ mm} \\
 &= 6 \text{ mm} \\
 &= 81 \text{ mm}
 \end{aligned}$$

$$\begin{aligned}
 M &= 917.3045682 \times \frac{4}{0.081} \\
 Z &= \frac{18.575 \text{ kNm}}{\pi d^3} \\
 d^3 &= \frac{98174.77042 \text{ mm}^3}{18575417.506 \times \frac{32}{\pi}} \\
 d &= 756830.5961 \text{ mm}^3 \\
 &= 91.1310191 \text{ mm} \\
 &= \text{or say} \\
 &= 100 \text{ mm}
 \end{aligned}$$

Dia of pin provided  
Design of triangular plate connected to base plate for holding the pin

From shear consideration  
 Resultant Load acting on the plate

$$\begin{aligned}
 &= 0.5 f_y A \\
 &= \frac{458.6522841 \times 1000}{250 \times 2} \\
 &= 917.305 \text{ kN}
 \end{aligned}$$



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1834.609136 sqmm

25 mm

1834.609136

25

73.38436546

75 mm

244.410 kN

2

122.205 kN

122.205

122.205 x

1000

250

977.64 sqmm

25

977.64

25

39.1056

75 mm

>

60

mm

Let us provide min. width around pin

Width of connecting plate to strut of frame

Maximum of following two

Resultant Force action on the plate

Member force from staad

Force acting on connecting plate

Tension Capacity of Plate

917.305 kN

917.280 kN

917.305 kN

0.900 fuAn

1.250

0.720 fuAn

917.305 kN

917.305 x

1000

x

410

x

50

62.14800598

Min. width of connecting plate to be provided

dia. of pin

100

+ twice of width around pin

+ (

2

x

60 )

i)  
ii)

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### Shear Check

Shear on plate from STAAD  
0.5 fyp A

Shear capacity of plate

=	220	mm
or say	220	x 50 mm
=	6.69	kN
=	6.69	
=	0.5	x 250.000 x 220 x 50
=	1375000	N
=	1375	kN > 6.69 kN

### Connection between connecting plate & tube

External width of Tube /Pipe

Thickness of pipe

Angle of strut with vertical

Horizontal length of connecting plate at face of tube

=	120 mm
=	8 mm
=	63.886 Degree = 1.115021
=	220
=	Cos 1.115021046
=	499.82
=	500 x 12

Let us provide a flange plate of size

Let us weld the flange plate to the tube by providing fillet weld

Welding between circular plate & connecting plate

i) Length of weld by welding on both sides

2 x ( 499.82 + 50 )

1099.64 mm

Thickness of weld required

917.305 x 1000

1099.6

7.542371159 mm

Let us provide weld thickness

8 mm

### Welding of supporting triangular plate with base plate

Required Length of connection of plate with base

75 ) +

Let us provide

Edge distance of plate on both sides

Length of base plate

250 mm

250 mm

50 mm

350 mm

350 x 25

8 mm

Let us provide

Let us take size of fillet weld



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Effective length of weld =  $\frac{2}{750} \times (350 + 25)$  mm

Force on the weld is maximum of the following two values

i) VR

$$= V = \frac{499.44^2}{249440.3136} + \frac{244.41^2}{1.265625} = 59736.2481 + 1.265625$$

$$= V = 309177.8273$$

$$= 556.0376132 \text{ kN}$$

ii) VR

$$= V = \frac{825.975^2}{682234.7006} + \frac{399.015^2}{0.2916} = 159212.9702 + 0.54^2$$

$$= V = 841447.9625$$

$$= 917.3047271 \text{ kN}$$

$$\frac{0.7}{s} \times \frac{158}{4} = \frac{Lw}{s} = \frac{917.3047271 \times 1000}{917.3047271 \times 158} = \frac{917.30473}{1000} \times \frac{750}{158}$$

Let us provide size of fillet weld =  $\frac{2.765 \text{ mm}}{8 \text{ mm}}$

Connection between connecting plate & flange plate

External width of Tube / Pipe = 240 mm

Thickness of pipe = 8 mm

Let us provide a flange plate of size 240 x 16

Let us weld the flange plate to the tube by providing fillet weld

Welding between circular plate & connecting plate

i) Length of weld by welding on both sides =  $2 \times (580 \text{ mm})$

Thickness of weld required =  $\frac{917.305 \times 1000}{158 \times 580}$

Let us provide weld thickness =  $\frac{14.29981555 \text{ mm}}{15 \text{ mm}}$

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### Design of blotted connection of parts of top member

Member force in member no

Load Comb.	Fx	8871
DL + LL	-79.22	
Design Load	-118.83	
DL + WL	-1034.64	
Design Load	-1551.96	

Fy	Fz
-105.57	17.33
-158.355	25.995
-98.16	173.92
-147.24	260.88

### Design of connection

Shear force is maximum of two

$$i) \quad P = V = -118.83^2 + 121.64 \text{ kN} = 25.995^2$$

$$ii) \quad P = V = -1551.96^2 + 1573.734 \text{ kN} = 260.88^2$$

Let us take

$$P = 260.88 \text{ kN}$$

### 1 Tension Capacity

Tension capacity

$$i) \quad T_b = \frac{0.9}{1.25} \times \frac{f_{ub}}{A_n} = \frac{0.9}{1.25} \times \frac{561}{900} \times 1000 = 561$$

Let us take diameter of bolts

Anb net shear area at the threads

Asb nominal shank area

$$d = 9.8 \text{ mm}$$

$$T_b = \frac{0.9}{1.25} \times \frac{706.858}{900} \times 1000 = 561$$

Tension capacity of each bolt

Number of bolts required

Let us provide

$$T_b = \frac{158.355}{363.528} = 0.435606061 \times 4 = 1.742424242$$



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$$\begin{aligned}
 \text{Bearing force on each bolt} &= \frac{158.355}{8} \\
 &= 19.794375 \text{ kN} \\
 \text{sum of thickness of plates} &= \frac{19.794375}{2.5} \times 1000 \times 30 \times 900 \\
 &= 0.5865 \text{ mm} \\
 \text{Thickness of each plate} &= \frac{0.5865}{2} \\
 &= 0.29325 \text{ mm} \\
 \text{Let us provide plate thickness} &= 12 \text{ mm}
 \end{aligned}$$

Bolts with Shear and Tension

$$\frac{V}{V_{sd}} = \frac{T_e}{T_{nb}} = \frac{2}{2} = 1$$

$$\begin{aligned}
 V &= \text{Applied factored shear} = 196.717 \\
 V_{sd} &= \text{Design shear capacity} = 233.21 \\
 T_e &= \text{Externally Applied factored tension} = 158.355 \\
 T_{nd} &= \text{Design tension capacity} = 363.528
 \end{aligned}$$

$$\begin{aligned}
 &= \frac{196.717}{233.21} + \frac{158.355}{363.528} \\
 &= 0.712 + 0.19 \\
 &= 0.902 < 1 \\
 &\text{so OK}
 \end{aligned}$$

#### 4 Welding of supporting plate with base plate

no and Size of connecting plates at base

Minimum size of fillet weld

Thickness of thicker part

upto 10

>10 upto 20

>20 upto 32

>32 upto 50 mm

Let us take size of fillet weld

Effective length of weld

$$\begin{aligned}
 &= \frac{2}{2} \times 400 + 20 \text{ mm} \\
 &= 400 + 20 = 420 \text{ mm}
 \end{aligned}$$

Minimum weld size should not be more than the thickness of thinner part

$$\begin{aligned}
 &= 3 \text{ mm} \\
 &= 5 \text{ mm} \\
 &= 6 \text{ mm} \\
 &= 8 \text{ mm} \\
 &= 12 \text{ mm} \\
 &= 2 \times 20 = 40 \text{ mm}
 \end{aligned}$$



  
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Force on the weld is maximum of the following two values

i)  $V_R$

$$= V \frac{118.83^2}{25076.30603} + \frac{158.355^2}{675.74003} + 25.995^2$$

$$= V \frac{14120.569}{25076.30603} + \frac{25076.30603}{675.74003} + 25.995^2$$

$$= V \frac{39872.615}{25076.30603} + \frac{25076.30603}{675.74003} + 25.995^2$$

$$= 199.68128 \text{ kN}$$

ii)  $V_R$

$$= V \frac{1551.96^2}{21679.6176} + \frac{147.24^2}{68058.374} + 260.88^2$$

$$= V \frac{2408579.8}{21679.6176} + \frac{21679.6176}{68058.374} + 260.88^2$$

$$= V \frac{2498317.8}{21679.6176} + \frac{21679.6176}{68058.374} + 260.88^2$$

$$= 1580.6068 \text{ kN}$$

Let us provide size of fillet weld

s	0.7	x	Lw s	=	1580.6068 kN
		158 x	1580.606793 x	1000	
		4 x	0.7 x	158 x	840
		=	4.253 mm		
		=	8 mm		

Let us provide size of fillet weld

Design of base plate

Let us take  $f_{yp}$   
 $f_u$

Tension capacity of Plate

Tnb	=	$\frac{0.9 f_u A_n}{\gamma_{m1}}$	410 N/sqmm
$f_u$	=	410 N/sqmm	
$A_n$	=	(b-ndh) t	
b	=	400 mm	
n	=	2	
dh	=	33 mm	
$A_n$	=	334 t	
$\gamma_{m1}$	=	1.25	
$T_e$	=	$\frac{1573.734}{1} = 1573.734 \text{ kN}$	
Tnb	=	$\frac{0.9 x}{1.25} = 334 \text{ t}$	
	=	98596.8 t	

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$$t = \frac{1573.734 \times 1000}{98596.8}$$

$$= 15.961309 \text{ mm}$$

$$\text{Let us provide if single plate}$$

$$\text{if double plate}$$

$$20 \text{ mm}$$

$$10 \text{ mm}$$

Design of bolts

Load at column base level:

P	=	-98.160	x	1.00	=	-98.16	KN
Mx	=	173.92	x	0.096	=	16.70	KNm
Mz	=	1034.64	x	0.096	=	99.33	KNm

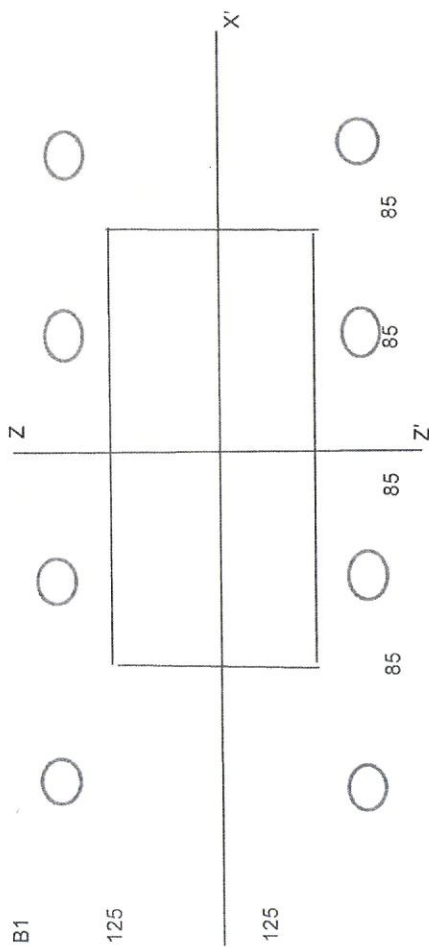
Let us provide 8 No. of bolts

B1

125

X

125



Max. load on bolt

$$= \frac{P}{N} + \frac{Mx \cdot z}{\sum(z^2)} + \frac{Mz \cdot x}{\sum(x^2)}$$

Sum(z<sup>2</sup>)

=	2	x	2	x	0.085
	2	x	2	x	0.17
	2	x	0	x	0
	0.1445	m <sup>2</sup>			

Sum(x<sup>2</sup>)

=	2	x	4	x	0.125
=	2	x	0	x	0
=	2	x	0	x	0
	0.125	m <sup>2</sup>			



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Max load on Bolt B1

$$= \frac{-98.16}{8} \quad \text{+/-} \quad \frac{16.70 \times 0.125}{0.125} \quad \text{+/-} \quad \frac{99.33 \times 0.1445}{0.1445} \quad \text{+/-} \quad 0.17$$

$$= \frac{-12.27}{121.279779} \quad \text{+/-} \quad \frac{16.8963}{-145.82} \quad \text{+/-} \quad 116.8535$$

C.S.area of H.T.Bolts required

$$= \frac{1.5 \times 145.81978 \times 1000}{1 \times 720} \times 1.25$$

Net stress area of

$$= \frac{379.7390074 \text{ Sq.mm}}{561 \text{ Sq.mm}} > \text{So O.K.} \quad 379.739 \text{ Sq.mm}$$

Bond stress for  
Length of bolts:

$$M \quad 30 \quad \text{Concrete} = \frac{1.5}{145.820} \times 1000 = 1.5$$

Minimum length

$$= \frac{1547.196 \text{ mm}}{24 \times 720 \text{ mm}} = 30$$

Let us provide length of bolts



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# Design of bloted connection of parts of top member

8871

Member force in member no

Load Comb.

DL + LL	Service Load	Fx
		332.96
	Design Load	499.44
DL + WL	Service Load	550.65
	Design Load	825.975

Fy	Fz
162.94	-0.75
244.41	-1.125
266.01	0.36
399.015	0.54

## Design of connection

Shear force is maximum of two

$$i) \quad P = V = 499.44^2 + -1.125^2 = 499.441 \text{ kN}$$

$$ii) \quad P = V = 825.975^2 + 0.54^2 = 825.975 \text{ kN}$$

$$P = 825.975 \text{ kN}$$

$$fyp = 720 \text{ N/sqmm}$$

$$fu = 900 \text{ N/sqmm}$$

Let us take

## 1 Tension Capacity

Tension capacity	Tb	=	i)	0.9	fub	An	
			ii)	1.25	fyb	An	
				1.1			
Let us take diameter of bolts			d	=	24	mm	
Anb net shear area at the threads	Grade		9.8	=	352.503	sqmm	
Asb nominal shank area	Tb	=		=	452.389	sqmm	
				0.9	x	900	x
				1.25	x	1000	
				228.422	N		
	Tb	=		1.25	x	720	x
				1.1	x	1000	
				288.412	kN		
				228.422	kN		
Tension capacity of each bolt							
				399.015			
				228.422			
Number of bolts required				1.746832617			

352.5

352.5

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## 2 Shear Capacity

shear capacity of bolts = 
$$\frac{f_u}{\sqrt{3}} (n_a A_{nb} + n_s A_{sb}) \times \beta_{ij} \beta_{1g} \beta_{pkg}$$

$n_a$  is no. of shear planes with threads

$n_s$  is no of shear planes without threads

Let us take diameter of bolts

$d = 24 \text{ mm}$

$p = 3 \text{ mm}$

Anb net shear area at the threads = 
$$\frac{\pi}{4} (d - 0.9382p)^2$$

= 352.503 sqmm

Asb nominal shank area

= 452.389 sqmm

$V_{nsb} = \frac{900}{1.732} \times 352.503$

= 183171.305 N

$V_{sb} = \frac{183171.305}{1.25}$

= 146.537 kN

Shear capacity of each bolt

Number of bolts required

=  $\frac{825.975}{146.537}$

= 5.636632226

Required number of bolts

= 5.636632226

Let us provide

= 8 bolts

## 3 Bearing Capacity

Bearing capacity of bolts

= 2.5 kgt fu

For over size or short slotted holes  $V_{npb}$  is to be multiplied by 0.7

For long slotted holes,  $V_{npb}$  is to be multiplied by 0.5

$d$  is the nominal diameter of the bolt = 24

kb for min edge distance of 1.5 x bolt hole diameter and minimum pitch is 2.5 x bolt hole diameter

Diameter of hole upto 14 mm dia bolt = dia of bolt + 1 mm



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for

> 14mm and upto 26mm = dia of bolt + 2 mm  
 > 26mm = dia of bolt + 3 mm

24 mm dia bolt = 24 + 26 = 39 mm  
 Min edge distance = 1.5 x 26 = 65 mm  
 Min pitch distance = 2.5 x 26 = 65 mm  
 kb = 0.5

Bearing force on each bolt =  $\frac{399.015}{8}$

sum of thickness of plates =  $\frac{49.876875}{49.876875} \times 1000$

Thickness of each plate =  $\frac{1.84729167}{1.84729167} \times 1000$

Let us provide plate thickness = 10 mm

Bolts with Shear and Tension

V	2	+	Te	2	<	1
Vsd			Tnb			

V = Applied factored shear = 103.247  
 Vsd = Design shear capacity = 146.537  
 Te = Externally Applied factored tension = 49.876875  
 Tnd = Design tension capacity = 228.422

$\frac{103.247}{146.537} + \frac{49.876875}{228.422} = 0.496 + 0.048 = 0.544 < 1$   
 so OK

4 Welding of supporting plate with base plate

no and Size of connecting plates at base  
 Minimum size of fillet weld

Thickness of thicker part Minimum weld size should not be more than the thickness of thinner part

25 mm

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upto 10 mm  
 >10 upto 20 mm  
 >20 upto 32 mm  
 >32 upto 50 mm  
 Let us take size of fillet weld  
 Effective length of weld

Force on the weld is maximum of the following two values

$$V_R = V \cdot 499.44^2 + 244.41^2 + 1.125^2$$

$$= V \cdot 249440.31 + 59736.2481 + 1.265625$$

$$= V \cdot 309177.83$$

$$= 556.03761 \text{ kN}$$

ii)  $V_R = V \cdot 825.975^2 + 399.015^2 + 0.54^2$

$$= V \cdot 682234.7 + 159212.9702 + 0.2916$$

$$= V \cdot 841447.96$$

$$= 917.30473 \text{ kN}$$

Let us provide size of fillet weld

$$s = \frac{0.7 \times 917.30473 \text{ kN}}{158 \times 0.7 \times 1000} = \frac{917.3047271 \times 1000}{158 \times 0.7 \times 1000} = 8.19 \text{ mm}$$

Let us provide size of fillet weld

### 5 Design of base plate

Let us take  
 $f_{yp}$   
 $f_u$

Tension capacity of Plate

Tnb =  $\frac{0.9 f_u A_n}{\gamma_{m1}}$

$f_u = 410 \text{ N/sqmm}$

$A_n = (b - ndh) \cdot t$

$b = 400 \text{ mm}$

$n = 2$

$dh = 26 \text{ mm}$

$A_n = 348 \text{ t}$

$\gamma_{m1} = 1.25$



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$$T_e = \frac{825.975}{1} = 825.975 \text{ kN}$$

$$T_{nb} = \frac{410 \times 348 \text{ t}}{1.25}$$

$$t = \frac{102729.6 \text{ t}}{825.975 \times 1000}$$

$$= \frac{8.0402825 \text{ mm}}{16 \text{ mm}} = 8 \text{ mm}$$

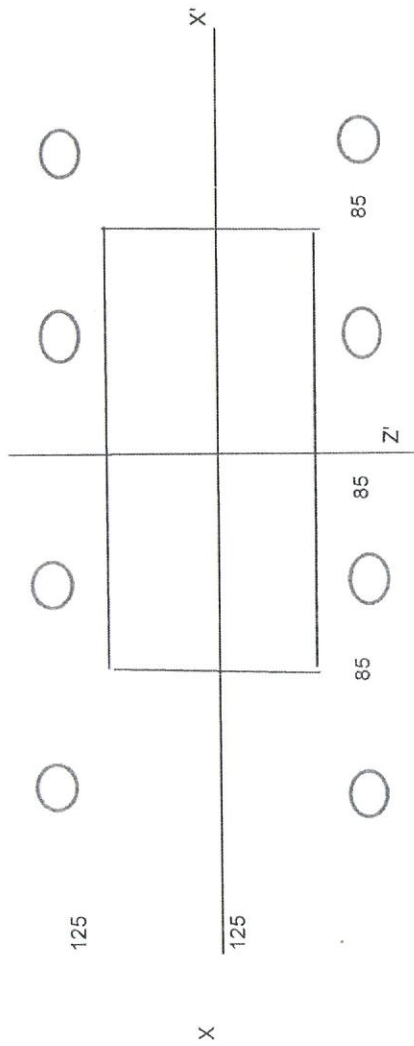
Let us provide if single plate if double plate

Design of bolts

Load at column base level:

P	=	266.010	x	1.00	=	266.01	KN
Mx	=	0.36	x	0.1575	=	0.06	KNm
Mz	=	550.65	x	0.1575	=	86.73	KNm

$$\text{Let us provide } B1 = \frac{5.63663223 \text{ No. of bolts}}{Z}$$



Max. load on bolt	=	$\frac{P}{N}$	+	$\frac{Mx \cdot z +}{\text{Sum}(z^2)}$	+	$\frac{Mz \cdot x}{\text{Sum}(x^2)}$
	=	$\frac{2}{2}$	x	$\frac{2}{2}$	x	$\frac{0.085}{0.17}$
Sum(z <sup>2</sup> )		2	x	2	x	0
		2	x	0	x	2



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Max load on Bolt B1

$$= \frac{266.01}{8} \times \frac{0.125}{0.06} \times \frac{0.125}{86.73} \times \frac{0.17}{0.1445}$$

C.S.area of H.T.Bolts required

$$= \frac{1.5 \times 68.837655 \times 1000}{1 \times 720} \times 1.25$$

Net stress area of

	=	179.2647289 Sq.mm
24 mm dia.bolts	=	179.2647 Sq.mm
	>	452.389 Sq.mm

Bond stress for  
Length of bolts:


M	30	Concrete	=	1.5	
				1.5 x	68.838 x
					1000
		$\pi$	x	24 x	1.5

Minimum length

=	912.988 mm	24 x	24
=	=	576 mm	
=	=	925 mm	

Let us provide length of bolts



  
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DESIGN OF PILE  
&  
PILE CAP

2/4

### Calculation of load on piles

D=Depth of Pile Cap+Pedestal  
S=Spacing of piles  
N=No. of piles  
C/C distance of columns I & II in X direction  
CG of pile group from centre of column I in X dirn  
CG of pile group from centre of column II in X dirn  
Total vertical loads of Column I & II  
No. of piles in each row  
No. of rows

-347.19 kN

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Sum(x2)	=	2 x	2 x	1.8 <sup>2</sup>	
	+				
	+	0 x	2 x	0.9 <sup>2</sup>	
	=	12.96 +	0		
	=	12.96			
	=				
	=	-347.19 +	10.7 x	0.9	389.744 x 0.9
		6		4.86	12.96
	=	-57.865 +	1.981481481 +	27.065556	
	=	-28.817963			

Pile capacity

Maximum load allowed on each pile in group  
(As per clause no.5.2.3.5 of IS 2911-Part-II)

#### Design of pile cap

Let us take thickness of cap

Embedment length of piles

Clear overhang

For bottom reinforcement

For top reinforcement

Length of pilecap in Z direction

Dia of pile

Clear overhang

Length of Pile cap in X direction

Area of pile cap

Volume of pile cap

Length of Pedestal in X direction

Length of Pedestal in Z direction

Thickness of Pedestal

Volume of Pedestal

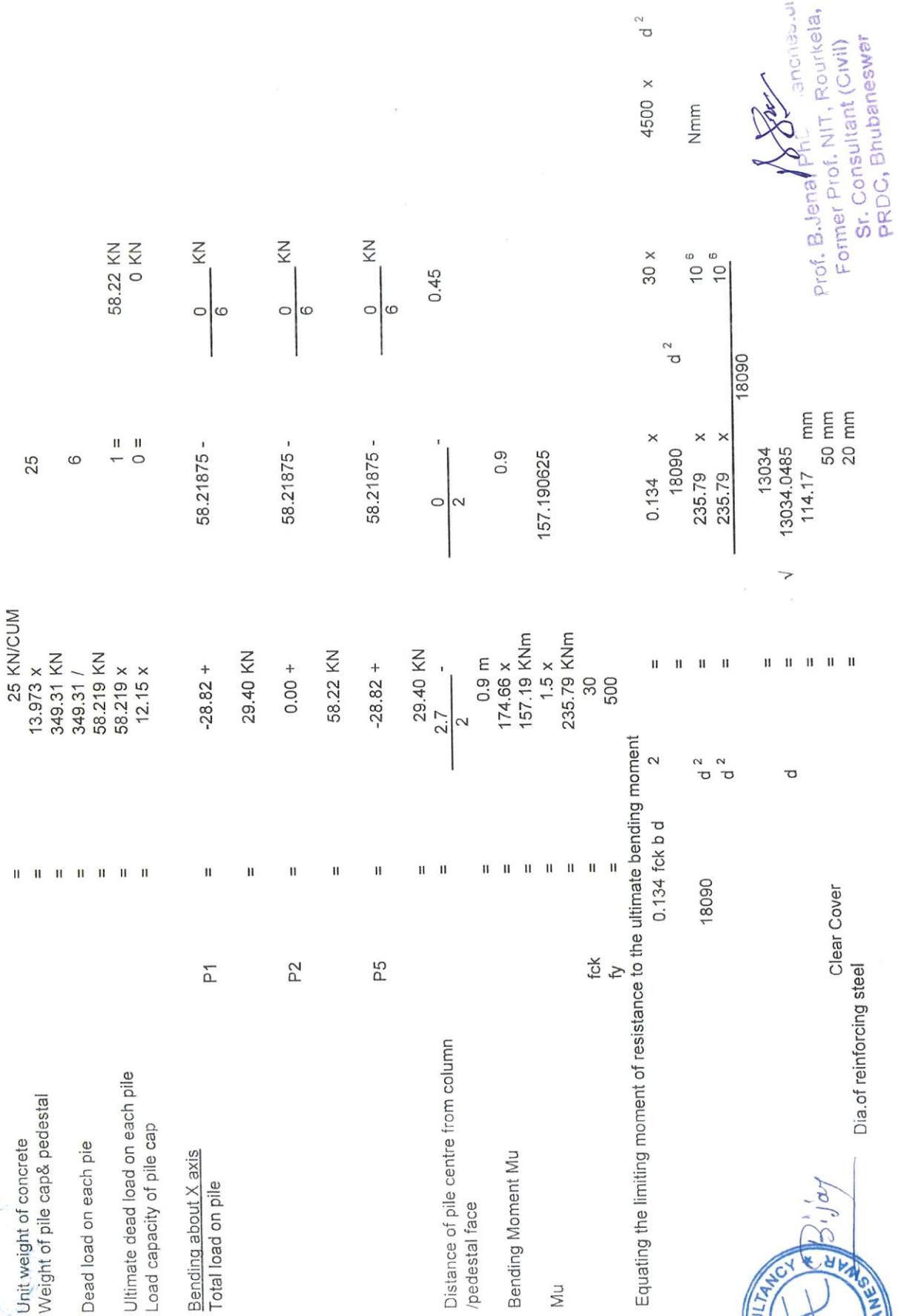
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0 x

0.000 Cum





Overall depth required

Let us provide  
No. of bars provided  
Ast provided

114.17 +  
219.17  
1045.00 mm  
0.48 x  
0.48 x  
501.6 mm  
mm C/C  
4350 /  
30,000 =  
25 x  
7857.14 Sq.mm.  
0.87 fy Ast  
0.362 fck b  
69,938 mm  
X u, max  
0.87 fy Ast (d-0.416X u)  
235.79 KNm  
Hence O.K.

75 + 20 + 20 / 2

(decide according to moments)  
d  
1045.00

150 )  
25 Nos.  
314.2857

Hence under-reinforced

Bending about Z axis  
Total load on pile

P1 = -28.82 +  
29.40 KN  
-28.82 +  
29.40 KN  
1.8 m  
0.6 x  
300 mm  
300 mm  
4.5 -  
2  
58.22 -  
58.22 -  
0.3 m  
all around the column  
0 -  
2  
0.45

Spacing of piles  
Size of column  
Width of pedestal  
Depth of pedestal  
Distance of pile centre from column / pedestal face

Bending Moment Mu

Mu

fck  
fy

Equating the limiting moment of resistance to the ultimate bending moment

0.134 fck b d  
2  
10854  
d<sup>2</sup>

0.134 x  
10854  
158.764 x  
d<sup>2</sup>

30 x  
10<sup>6</sup>

Nmm

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2700 x d







	$d^2$	$=$	$\frac{158.764 \times 10^6}{10854}$
	$d$	$=$	$\sqrt{\frac{14627}{14627.2572}}$
		$=$	$\frac{120.943}{50}$
		$=$	$\frac{20}{20}$
		$=$	$\frac{120.943 + 200.943}{1065}$
		$=$	$\frac{0.48}{0.48}$
		$=$	$\frac{0.48}{0.48}$
		$=$	$\frac{511.2}{150}$
		$=$	$\frac{2700}{18.333}$
		$=$	$\frac{14}{4400.00}$
		$=$	$\frac{0.87 f_y A_{st}}{0.362 f_{ck} b}$
		$=$	$\frac{65.275}{0.87 f_y A_{st}(d-0.416X_u)}$
		$=$	$\frac{1986.44}{158.76}$
		$=$	$\frac{1045.00}{2}$
		$=$	$\frac{1045}{1045.000}$
		$=$	$\frac{4500}{1200}$
		$=$	$\frac{600}{174.66}$
		$=$	$\frac{45.120}{1.5}$
		$=$	$\frac{67.679}{67.679}$

Clear Cover

Dia. of reinforcing steel

Overall depth required

provided

X u, max

Let us provide 20 mm dia. Bars @

No. of bars provided

Ast provided

X u

X u

MuR

### 1) Check for Oneway Shear (parallel to X direction)

Critical Section is at effective depth from column/pedestal face

Distance of critical section from C.G.

Effective depth at critical section

Width of footing at critical section

Distance of outer edge of pile from centre

Distance of inner edge of pile from centre

Shear Force at the critical section

Su



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Nominal Shear Stress

Percentage of tensile steel at the critical section

From IS 456:2000, Design shear strength

$$\begin{aligned}
 &= \frac{67.679 \times 1000}{4500 \times 1045} \\
 &= \frac{0.014 \text{ N/Sq.mm}}{7857.143} \times 100 \\
 &= 0.167 \% \\
 &= 0.15 \\
 &= 0.25 \\
 &= 0.37 \\
 &= 0.167 = \\
 &= 0.290 + \frac{0.08 \times 0.017}{0.1} \\
 &= 0.304 \text{ N/Sq.mm} > 0.014 \text{ N/Sq.mm} \\
 &\text{Hence O.K.}
 \end{aligned}$$

# 1) Check for Oneway Shear(parallel to Z direction)

Critical Section is at effective depth from column/pedestal face  
Distance of critical section from C.G.

Effective depth at critical section  $d$   
Width of footing at critical section  $b$   
Distance of outer edge of pile from centre  
Distance of inner edge of pile from centre  
Shear Force at the critical section  $S$

Nominal Shear Stress

Percentage of tensile steel at the critical section

$$\begin{aligned}
 &= \frac{0 + 1065}{2} \\
 &= 1065 \text{ mm} \\
 &= 1065.000 \text{ mm} \\
 &= 2700 \text{ mm} \\
 &= 2100 \text{ mm} \\
 &= 1500 \text{ mm} \\
 &= 58.80 \times (2100 - 600) \\
 &= 58.802 \text{ KN} \\
 &= 1.5 \times 58.80157 \\
 &= 88.202 \text{ KN} \\
 &= \frac{88.202 \times 1000}{2700 \times 1065} \\
 &= \frac{0.031 \text{ N/Sq.mm}}{4400.000} \times 100 \\
 &= 0.153 \% \\
 &= 0.15
 \end{aligned}$$



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From IS 456:2000, Design shear strength

$$\begin{aligned} 0.25 & 0.36 \\ 0.5 & 0.48 \\ 0.153 & = \end{aligned}$$

$$\begin{aligned} 0.360 + & \frac{0.12 \times}{0.25} - 0.097 \\ 0.313 \text{ N/Sq.mm} & > 0.031 \text{ N/Sq.mm} \\ \text{Hence O.K.} & \end{aligned}$$



  
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### & Rectangular pile cap

Calculation

Fx	172.74 KN
Fy	-158.1 KN
Fz	757.36 KN
Mx	-4.99 KNm
My	0 KNm
Mz	193.661 KNm
TMx	-870.964 KNm
TMz	1.150 m
D	1.8 m
S(X direction)	1.8 m
S(Z direction)	1.8 m
N	6 Nos
ez	193.661
	<hr/>
	-158.1
	-1.225

$$\begin{array}{r} \text{ex} \\ -870.964 \\ \hline -158.1 \\ 5.509 \end{array}$$

D=Depth of Pile Cap  
S=Spacing of piles

 $N = \text{No. of piles}$ 

C/C distance of columns I &amp; II in X direction

CG of pile group from centre of column I in X dirn


CG of pile group from centre of column II in X dirn

Total vertical loads of Column I &amp; II

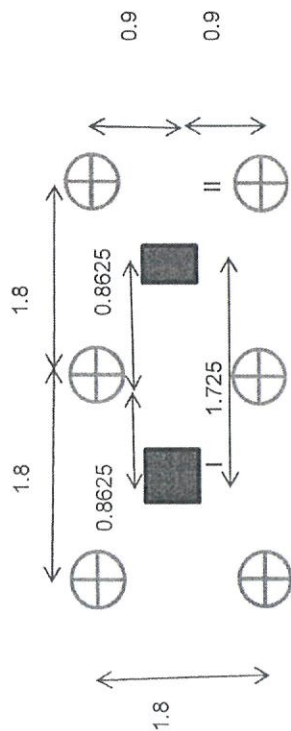
No. of piles in each row

No. of rows

$$\begin{array}{rcl} 1.725 & & \\ 0.8625 & & \\ 0.8625 & & \\ -158.1 & + & -163.75 \\ \hline & & = \end{array} \quad \begin{array}{r} -321.85 \text{ kN} \\ \\ \\ \\ \hline \end{array}$$

  
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Eccentricity of load of column I from CG of pile group in x direction	0.8625	-	5.509
ex	=		
	=	-4.6465	
Mz due to load of Column I due to eccentricity from CG of pile group	-158.1	x	-4.6465
	=	734.61165	
Eccentricity of load of column II from CG of pile group in x direction	0.8625	+	-7.527
ex	=		
	=	-6.6645	
Mz due to load of Column II due to eccentricity from CG of pile group	-163.75	x	-6.6645
	=	1091.31188	
Total Mz	=	734.612 -	1091.312
	=	-356.7	KNm
Total Mx	=	193.661 +	146.0705
	=	339.7315	KNm

Max. load on pile	$\frac{P}{N}$	+	$\frac{Mx \cdot z}{\text{Sum}(z^2)}$	+	$\frac{Mz \cdot x}{\text{Sum}(x^2)}$
	=				
	=	1 x	3 x		0.9 <sup>2</sup>
	=	1 x	3 x		0.9 <sup>2</sup>
	=	2.43 +	2.43		
	=	4.86			



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Sum(x2)	=	2 x	1.8 <sup>2</sup>		
+					
+		2 x	0.9 <sup>2</sup>		
=					
=		0			
		12.96 +			
		12.96			
=		-321.85 +	339.7315 x	0.9	356.7 x 0.9
		6	4.86	+	12.96

24.77083333

Pile capacity  
Maximum load allowed on each pile in group  
(As per clause no.5.2.3.5 of IS 2911-Part-III)

### Design of pile cap

Let us take thickness of cap	=	1150 mm
Embedment length of piles	=	50 mm
Clear cover	=	75 mm
For bottom reinforcement	d	1150 - 75 - 20 = 1055 mm
		20
For top reinforcement	d	1065 mm
		1065 - 20 = 1045 mm
		20

Length of pilecap in Z direction  
Dia of pile  
Clear overhang  
Length of Pile cap in X direction  
Area of pile cap

Volume of pile cap

Length of Pedestal in X direction  
Length of Pedestal in Z direction  
Thickness of Pedestal  
Vol.of pedestal

Unit weight of concrete



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Weight of pile cap & pedestal

=  
= 13.973 x  
= 349.31 KN  
= 349.31 /  
= 58.219 KN  
= 58.219 x  
= 12.15 x

25

Dead load on each pile

6

Ultimate dead load on each pile

1 =  
0 =

58.22 KN  
0 KN

Load capacity of pile cap

Bending about X axis

Total load on pile

P1

58.21875 -

$\frac{0}{6}$  KN

P2

58.21875 -

$\frac{0}{6}$  KN

P5

58.21875 -

$\frac{0}{6}$  KN

Distance of pile centre from column / pedestal face

$\frac{2.7}{2}$  -

0.45

Bending Moment Mu

0.9

Mu

249.105125

fck

fy

30  
500

Equating the limiting moment of resistance to the ultimate bending moment

$0.134 f_{ck} b d$

$d^2$

$x$

$4500$

$x$

$30$

$x$

$d^2$

$10^6$

$10^6$

$10^6$

$10^6$

$10^6$

$10^6$

$10^6$

$10^6$

$10^6$

$10^6$

$10^6$

$10^6$

$10^6$

$10^6$

$10^6$

$10^6$

Nmm

18090

0.134 x

18090

373.66 x

373.66 x

20655

20655.483

143.72 mm

50 mm

20 mm

143.72 +

248.72

1045.00 mm

Clear Cover

Dia. of reinforcing steel

Overall depth required

d provided



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20 + 20 / 2

(decide according to moments)



Let us provide  
 No. of bars provided  
 Ast provided

X u,max  
 20 mm dia. Bars @  
 150

(  
 4350 /  
 30.000 =  
 25 x  
 7857.14 Sq.mm.  
 0.87 fy Ast  
 0.362 fck b  
 69.938 mm  
 X u,max  
 fy Ast(d=0.416X u)  
 3472.221 KNm >  
 Hence under-reinforced  
 373.66 KNm  
 Hence O.K.

X u  
 MuR  
 =

34.04 +  
 92.26 KN  
 34.04 +  
 92.26 KN  
 1.8 m  
 0.6 x  
 300 mm  
 300 mm  
 4.5  
 2

1.8 m  
 184.52 x  
 332.14 KNm  
 1.5 x  
 498.21 KNm  
 30  
 500

Equating the limiting moment of resistance to the ultimate bending moment  
 0.134 fck b d  
 2  
 10854  
 d<sup>2</sup>  
 d<sup>2</sup>  
 d

0.134 x  
 10854  
 498.210 x  
 498.210 x  
 10854  
 45901  
 45901.0733  
 ✓

Spacing of piles  
 Size of column  
 Width of pedestal  
 Depth of pedestal  
 Distance of pile centre from column /pedestal face

Bending about Z axis  
 Total load on pile

Bending Moment Mu  
 Mu

1883

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Clear Cover					
Dia. of reinforcing steel		214.245	mm		
Overall depth required		50	mm		
		20	mm		
		214.245 +			20 + 20 / 2
		294.245			
		1065	mm		
		0.48	x		
		0.48	x		
		511.2	mm		
		mm	C/C		
		150			
		(			
		2700 /			
		18.333 =			
		14 x			
		4400.00	Sq. mm.		
		0.87 fy Ast			
		0.362 fck b			
		65.275	mm		
		X u, max			
		fy Ast (d=0.416X u)			
		1986.44 KNm			
		>			
		498.21 KNm			
		Hence O.K.			
		Hence under-reinforced			

1) Check for Oneway Shear (parallel to X direction)

Critical Section is at effective depth from column/pedestal face  
Distance of critical section from C.G.

Effective depth at critical section  
Width of footing at critical section  
Distance of outer edge of pile from centre  
Distance of inner edge of pile from centre  
Shear Force at the critical section

Nominal Shear Stress

Percentage of tensile steel at the critical section

	$\frac{0 + 1045}{2}$	1045.00	
	1045 mm		
	1045.000 mm		
	4500 mm		
	1200 mm		
	600 mm		
	276.78 x	(	
	71.502 KN		
	1.5 x		
	107.254 KN		
	Su/b'd'		
	107.254 x		
	4500 x		
	0.023 N/Sq. mm		
	7857.143		
	4500	x	
	0.167	%	
	1045.000	x	100

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From IS 456:2000, Design shear strength

$$\begin{aligned} 0.15 & 0.29 \\ 0.25 & 0.37 \\ 0.167 & = \end{aligned}$$

$$\begin{aligned} 0.290 + & \frac{0.08 \times}{0.1} & 0.017 \\ 0.304 \text{ N/Sq.mm} & > 0.023 \text{ N/Sq.mm} \\ \text{Hence O.K.} & & \end{aligned}$$

# 1) Check for Oneway Shear(parallel to Z direction)

Critical Section is at effective depth from column/pedestal face  
Distance of critical section from C.G.

$$\begin{aligned} &= \frac{0 +}{2} 1065 \\ &= 1065 \text{ mm} \\ &= 1065.000 \text{ mm} \\ &= 2700 \text{ mm} \\ &= 2100 \text{ mm} \\ &= 1500 \text{ mm} \\ &= 184.52 \times (2100 - 600) \\ &= 184.522 \text{ KN} \\ &= 1.5 \times 184.52231 \\ &= 276.783 \text{ KN} \\ &= \frac{276.783 \times}{2700 \times} \frac{1000}{1065} \\ &= 0.096 \text{ N/Sq.mm} \\ &= \frac{4400.000}{2700 \times} \frac{1065.000}{100} \\ &= 0.153 \% \end{aligned}$$

Nominal Shear Stress

$$\begin{aligned} Su/b'd' &= \frac{276.783 \times}{2700 \times} \frac{1000}{1065} \\ &= 0.096 \text{ N/Sq.mm} \\ &= \frac{4400.000}{2700 \times} \frac{1065.000}{100} \\ &= 0.153 \% \end{aligned}$$

Percentage of tensile steel at the critical section

From IS 456:2000, Design shear strength

$$\begin{aligned} 0.25 & 0.36 \\ 0.5 & 0.48 \\ 0.153 & = \end{aligned}$$

$$\begin{aligned} 0.360 + & \frac{0.12 \times}{0.25} & -0.097 \\ 0.313 \text{ N/Sq.mm} & > 0.096 \text{ N/Sq.mm} \\ \text{Hence O.K.} & & \end{aligned}$$

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# DRAWINGS





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### PILE CAP DETAILS

A  
TRANSVERSE SECTION AT Y-Y

HEAD SHOWING CABLES