Fisheries and Oceans    Pêches et Océans

# HARBOURS AND INFRASTRUCTURE BRANCH

## Newfoundland Region



Canada

# DESIGN OF A CONCRETE DECK FOR A ROUND TIMBER, PILE WHARF



Department of Fisheries and Oceans

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SUMMARY

The Department of Fisheries and Oceans is responsible for the provision and maintenance of fishing harbour facilities in Newfoundland. Wharf construction accounts for a large portion of the funds expended. There are currently several types of wharf designs used. Ballast filled crib wharves are the most common type and, although expensive, they are necessary in unsheltered harbours.

For more sheltered locations, treated, round timber pile wharves are a less expensive alternative. They require less material than crib wharves and do not need ballast. They are cheaper than steel pile wharves and are easier to maintain. This type of wharf can be utilized wherever bottom conditions allow driving of wooden piles.

The concrete deck for the wharf has to be poured onto formwork, as opposed to pouring onto the prepared ballast of a crib wharf. To facilitate this a steel floor decking should be used for the formwork. The Westeel-Rosco Hi-bond T-30-6 floor system, or similar material, is adequate for this purpose, with 3m X 3m panels between the pile caps. The formwork is left in place but does not contribute to the permanent strength of the structure.

The concrete strength should be specified as 30 MPa and the deck poured to a minimum of 200 mm increasing by a 2% grade for drainage. Adequate positive reinforcement consists of a grid of 20 mm bars at 165 mm centre to centre and a cover of 40 mm. Negative reinforcement should be provided by 20 mm bars at 165 mm, with a minimum of 50 mm cover, to provide strength in the vicinity of the pile caps. Cover will increase as the deck thickness increases to provide drainage.

Adequate inspection during construction should ensure that proper concrete placement techniques are adhered to. The deck should not be subjected to loadings until the concrete has sufficiently cured, and after curing the loading should be restricted to one MS-200 load.

This design is for a small to medium size wharf and is meant to provide general guidelines only. The design is a conservative one and considerable savings may be realized if a professional engineering detailed design was completed.

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## 1.0 INTRODUCTION

The Department of Fisheries and Oceans of Canada is responsible for a wide range of activities, including the provision of a national system of fishing and recreational boating harbours. More specifically the Harbours and Infrastructure Branch develops, maintains, and administers appproximately 500 locations in the Newfoundland Region. [Brown, 1982]

This report deals with the construction and maintenance of a particular type of wharf for some coastal areas of Newfoundland. In particular, the design of the concrete deck section of a wood pile wharf is presented. The design is a typical design and is not meant to be specific to any one particular location but can be easily adapted or modified for a wharf in any location where harbour conditions are similar.

The report gives background information on different types of wharves built in Newfoundland and gives justification for the selection of this particular type of wharf in some locations. A detailed design of the concrete deck is given as well as construction methods and procedures to ensure that the wharf has a maximum life span and low maintenance.

## 2.0 TYPES OF WHARVES

There are currently several different wharf designs used in Newfoundland. The particular type chosen depends on bottom conditions, degree of shelter, and number and size of vessels using the facility. These include: full crib timber wharf, crib and span timber wharf, steel pile wharf, and timber pile wharf. Also, there are many variations and combinations of the above types. The surface of a wharf is wood or concrete, preferably concrete, and the method of construction varies with the type of structure used for the foundation.

The full crib wharf is heavily ballasted with rock placed inside the crib and the concrete, essentially, poured onto the ballast. This type of wharf is expensive because of the large amounts of timber and ballast used and the labour required. However, for non-sheltered harbours this type is required because of its ability to withstand forces from heavy seas and ice and for the mooring shelter provided on the protected side of the structure.

The crib and span wharf is an attempt to reduce cost by eliminating some of the timber and ballast used in the construction. It consists of spaced cribs that are spanned by the concrete deck. Although the cost is reduced there are problems encountered in attaching the fendering system, and the protection on the sheltered side is reduced. [Stone]. A typical wharf crib is shown in Figure 1.

Where less protection is needed because of a better sheltered harbour a steel pile wharf can be constructed. The cost is less than that of a crib and span wharf. However, some problems arise with corrosion of the steel and sometimes a costly program of corrosion prevention is required. Also, maintenance of steel structures presents problems because most local contractors are not trained or do not have the equipment to easily repair a steel structure. [Stone]. Therefore, this type of wharf is probably not a practical alternative for small fishing harbours.

The cheapest alternative for a structure in sheltered harbours is a wooden pile wharf. Repairs are more easily facilitated because traditional methods of spiking and bolting the wood can be used and special equipment is not needed, as for steel structures. Also, as the water depth increases they become more economical, compared to a crib wharf. This is because the amount of wood required for a crib of given depth is much greater than that needed for piles, and the pile construction requires no ballast. Thus it would appear that a round timber pile wharf is a more practical alternative for general use in smaller and more isolated

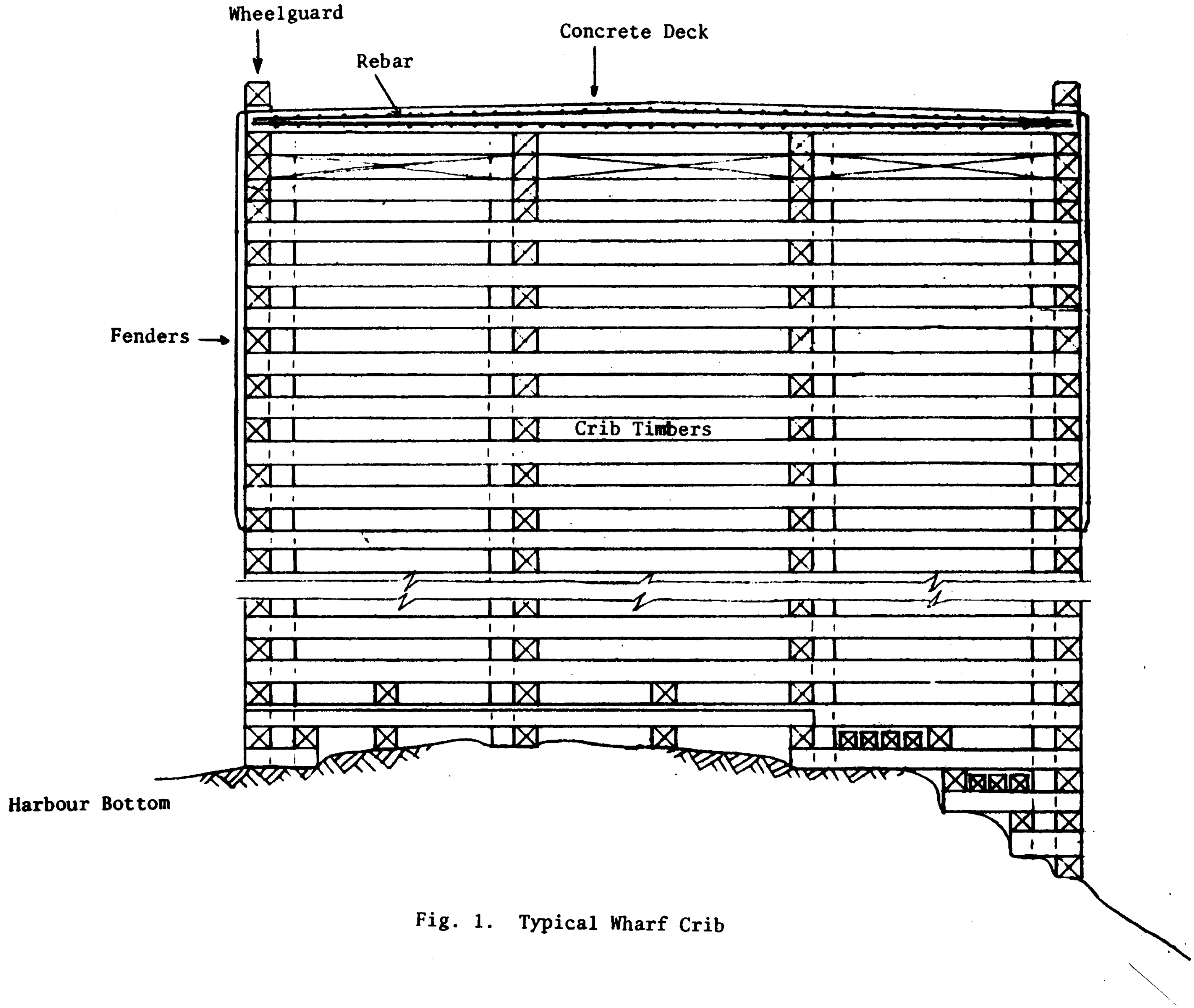


Fig. 1. Typical Wharf Crib

locations, where harbour conditions warrant. Fig. 2 and Fig. 3 shows a typical round timber pile wharf.

The piles are usually of southern pine species, imported from the United States and pressure treated at the Newfoundland Hardwoods Limited plant in Clarenville, Newfoundland. The piles are normally specified as No. 12 and are available in various lengths. The piles are placed by conventional methods, where bottom conditions permit. Fig. 2 shows a section of the wharf and Fig. 3 is a side view of the same structure.

This wharf projects perpendicularly from the shoreline, with both sides of the wharf available for mooring. When this type of wharf is constructed parallel to the shoreline it is termed a marginal wharf and only one side is available for mooring, the other side being adjacent to the shoreline.

The remainder of this report involves the design of a suitable concrete deck for a round timber pile wharf.

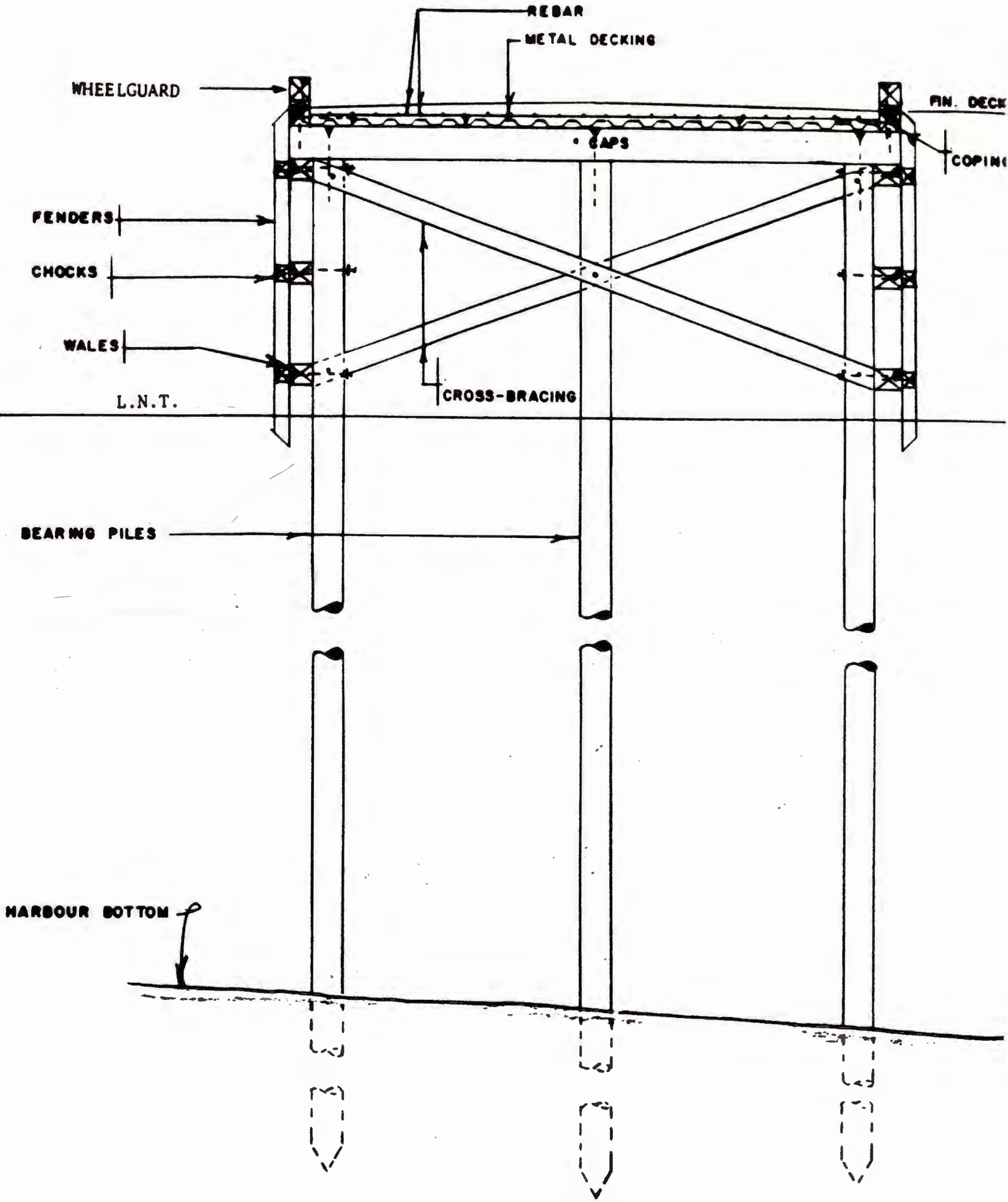


Fig. 2. Round Timber Pile Wharf

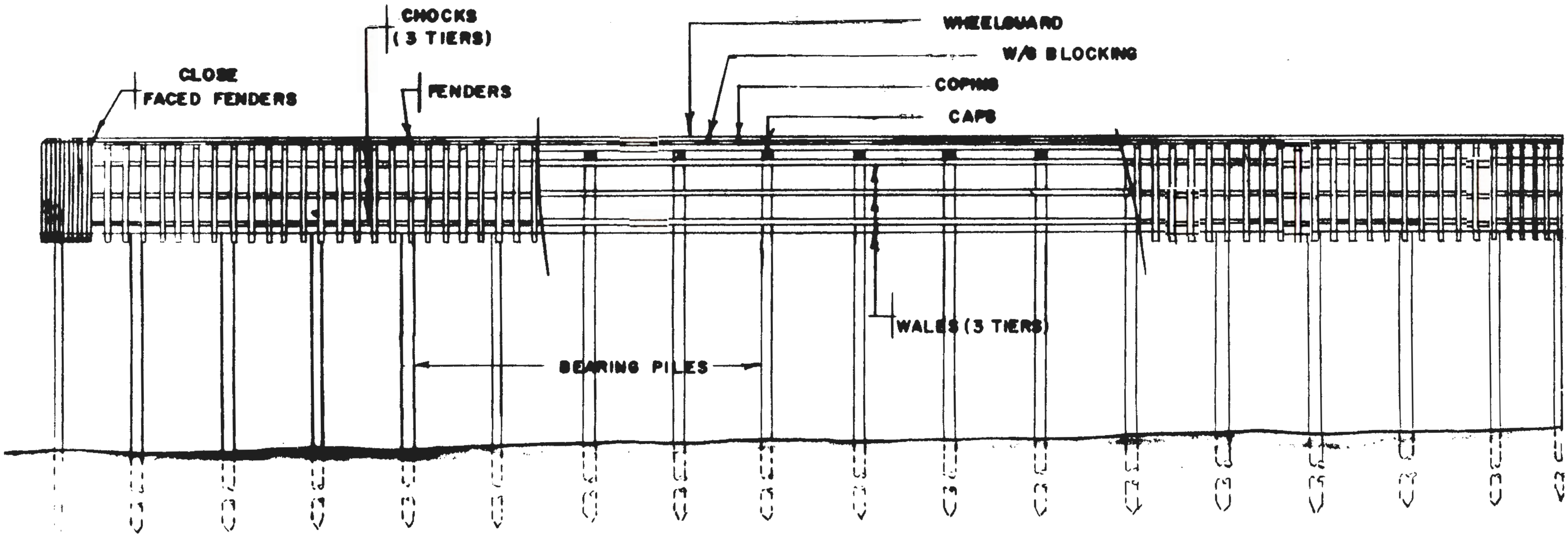


Fig. 3. Round Timber Pile Wharf

## 3.0 FORMWORK SELECTION

The formwork used to allow the placement of the wet concrete is a standard roll formed steel floor deck. This steel is available in various sizes and gauges and is ribbed or fluted in profile for extra strength. See Appendix A. Usually this steel decking acts with the concrete as a composite floor system. However, it must be noted that in marine construction, while the steel formwork is left in place, it is assumed to contribute no strength to the wharf deck after the concrete has cured. This is due to the high corrosive properties of the sea water and the expected relatively short life span of the decking. Hence, the reinforced concrete slab must be designed to handle all anticipated loads during the life of the structure, while the steel formwork has to withstand construction loads during placement of the wet concrete.

The steel formwork is available from several manufacturers and all have similar properties. For the purposes of this report the Westeel-Rosco Hi-bond system is chosen which is available locally and manufactured in Canada. The formwork is also available in several widths and any desirable length.

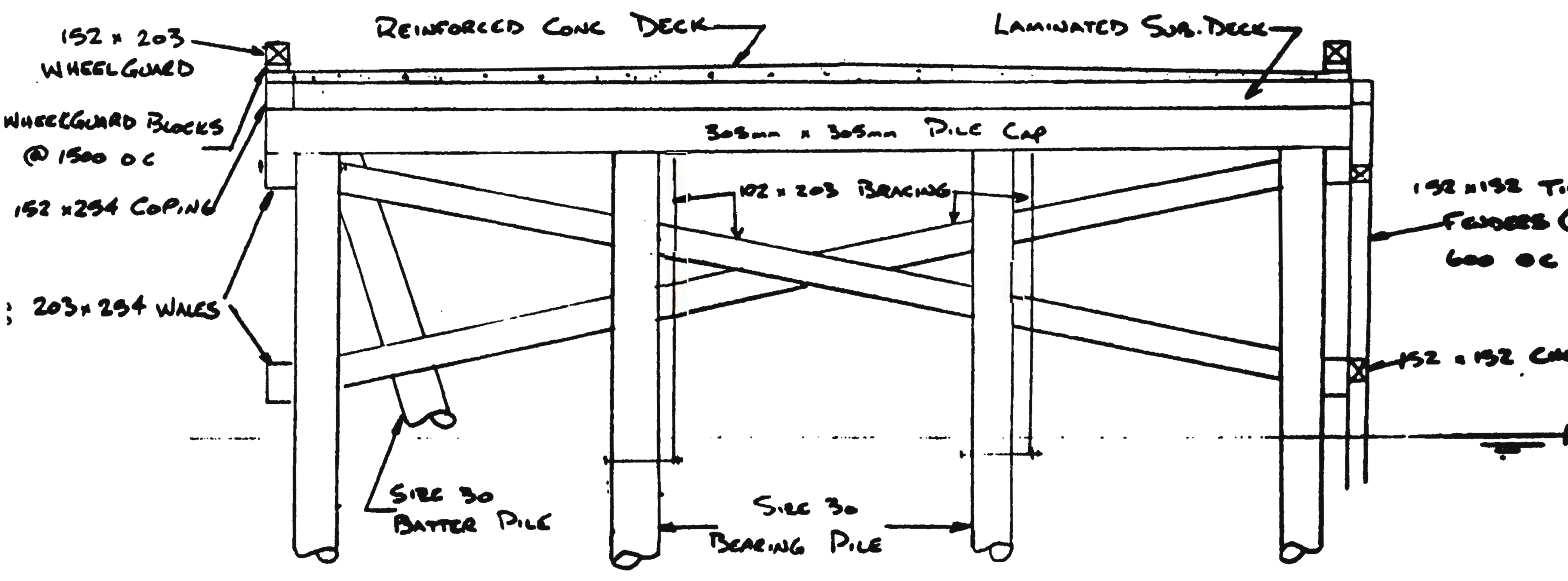


FIGURE # 4

TYPICAL SECTION - WOOD PILE WHARF

## 4.0 FORMWORK DESIGN

### 4.1 METHOD

The formwork is designed using the limit states design procedure. A specific type of formwork with known properties is selected. This formwork is then checked to determine whether or not it is sufficient to handle expected loadings. The span length and concrete weight are known and a slab thickness is assumed. The steel form must be able to withstand forces applied during pouring of the concrete. Specifically, deflection and stress of the steel formwork are checked.

### 4.2 RESULTS

A Westeel-Rosco Limited Hi-bond T-30-6 is selected from the selection chart. See Appendix A. A slab thickness of 200 mm is assumed. The span width is known to be 3 m. The modulus of elasticity of steel is 200 GPa.

A concrete density of 2400 kg/m^3 is used. The moment of inertia is 2610.6 x 10^3 mm^4. The deflection is calculated using CISC recommendations and is determined to be 9.5 mm due to the weight of the wet concrete. See Appendix B. The maximum permissable deflection is L/180 or 16.7 mm. Therefore, the selected formwork meets the requirements for deflection.

Stress is calculated using the applied moment, M and the section modulus s. M is calculated to be 5.3 kN.m due to the wet concrete and s = $60.18 \times 10^3$ $mm^3$ according to manufacturer's information. The calculated stress is $8.8 \times 10$ $kN/m^2$. Using a steel strength of 227 MPa the allowable stress is $14.1 \times 10$ $KN/m^2$. Therefore, the selected formwork meets the requirements for stress.

## 5.0 SLAB DESIGN

### 5.1 METHOD

The concrete slab must be designed to withstand the following loads:

1. Dead load, consisting of the self weight of the slab and the equipment that may be permanently placed on the wharf.

2. Live load, consisting of vehicle weight plus impact fraction, and snow loads.

The moment and shear is determined for maximum load combination and the slab is designed to withstand these loads. The amount of reinforcement used controls the maximum resistant moment while the thickness of the slab determines shear resistance. CAN/CSA-S6-88, Design for Highway Bridges, will be used as a design aid because a wharf deck subjected to traffic loads responds in a similar manner to a loaded highway bridge.

### 5.2 CALCULATION OF MOMENTS

The load due to the slab weight has been determined as 4.7 kPa. The Department of Public Works recommends a deck load of about 19 kPa [400 lb./ft^2] to account for permanent infrastructure. The factored dead load becomes

$$1.25\ [4.7\ kPa + 19.1\ kPa] = 30\ kPa$$

The live load due to snow is given by

$$s = Cs \times So$$

obtained from the supplement to the National Building Code. The ground snow load for St. Anthony is used because this represents the highest snow load for the island portion of the province. With a ground snow load of 5.7 kN/m^2 and a reduction factor for wind the factored snow load becomes

$$1.5[0.6 \times 5.7\ kN/m^2] = 5\ kPa$$

Therefore, the total uniform distributed load is equal to the dead load plus live load.

$$U.D.L. = 30\ kPa + 5\ kPa = 35\ kPa$$

For a wharf deck consisting of 4 equal spans and considering a one metre strip along the centre of the span the largest positive moment occurs in the outside panel.

$$M\ pos. = 0.077 \times 35kPa \times 3m \times 3m = 24kN.m$$

and the largest negative moment is

$$M\ neg. = 0.107 \times 35kPa \times 3m \times 3m = 34kN.m$$

These are the moments due to the uniformly distributed loads.

The point load is due to vehicle loading and is based on the requirement for the wharf to accommodate MS-200-77 loads. See Appendix B. The vehicle produces the largest moment in the slab when it is positioned at the centre of the span. The total weight of the vehicle is 450 kN. The point load due to one wheel becomes

$$P.L. = 0.15(450\ kN) = 67.5\ kN$$

according to clause 5.3.3.2.2 of CAN/CSA-S6-88. The impact factor is 0.30 and the total vehicle load becomes

$$(1.0+0.3) \times (67.5kN) = 88\ kN$$

This load is the design point load applied to the middle of the slab.

When the wharf is restricted to one MS-200 load the maximum moment occurs when the 88 kN load is applied to the centre of the outside span. The moments due to the vehicle become

$$M\ pos. = 0.200 \times 88\ kN \times 3m = 53\ kN$$

$$M\ neg. = 0.100 \times 88\ kN \times 3m = 26\ KN$$

The total positive moment is equal to the positive moment due to the uniformly distributed load and the positive moment due to the point load. (Vehicle load). The total positive moment is

$$M(+) = 24 \text{ kN.m} + 53 \text{ kN.m} = 77 \text{ kN.m}$$

and the negative moment becomes

$$M(-) = 34 \text{ kN.m} + 26 \text{ kN.m} = 60 \text{ kN.m}$$

These are the design moments for the longitudinal direction.

These moments have been calculated by assuming that the slab panels have one-way bending action. While this is not exactly the case , the procedure is simplified by making this assumption. The co-efficient method and the direct design method of two-way slab analysis is not applicable in this case. Extensive detailed analysis, such as the equivilent frame analysis, is necessary for a complete and accurate analysis using a software package would be helpful.

For the purposes of calculation only, it is assumed that the action is one way. The amount of transverse reinforcement provided will be equal to the amount of main reinforcement. Also, the amount of positive reinforcement will be the same for all panels. In both cases it will be equal to the amount necessary for the panel with the greatest moment. This makes it easy to put the rebar in place because it becomes a grid pattern, identical for each panel. It must be noted however, that this leads to a conservative design and the amount of reinforcing steel used could probably be reduced if a more accurate analysis was done.

## 5.3 SLAB THICKNESS

The slab thickness is calculated according to Table 29 of CAN/CSA-S6-88. For a 3m span the slab thickness is calculated to be 200 mm. A shear analysis will be performed later to determine if this is adequate, but the initial design will be based on the 200 mm thickness. The minimum cover for reinforcement for a slab is 50 mm, from Table 27 of CAN/CSA-S6-88. However, this can be reduced to 40 mm because of the protection provided by the steel formwork. The effective depth d, then becomes

$$d = 200 \text{ mm} - 40 \text{ mm} - 0.5 \times 20 \text{ mm} = 150 \text{ mm}$$

where 20 mm is the diameter of the steel reinforcement bar.

## 5.4 REINFORCEMENT

The reinforcement is calculated based on a one metre strip of concrete, a concrete strength of 30 MPa, and a steel strength of 400 MPa. The area of steel required has been calculated as 1740 mm^2/m. The actual amount used becomes 6, 20 mm bars at 300 mm^2 each, for 1800 mm^2 per metre strip. The calculated spacing is 167 mm. The actual spacing is specified as 165 mm.

For negative reinforcement the cover is 50 mm since there is no steel formwork to provide added protection. The effective depth d', then becomes 140 mm and the calculated reinforcement steel area is 1414 mm^2/m. For simplification however, the negative reinforcement provided shall be the same as the positive reinforcement. That is, 20 mm bars at 165 mm spacing.

5.5 SHEAR CHECK

The shear force V, in the slab can be approximated by considering a one metre strip of the slab. The shear force is due to the uniform load and the point load. The maximum shear force occurs at a distance from the support equal to d, the effective depth of the slab. It is also assumed that the point load due to the vehicle is at the same distance d, from the support.

$$V = w \times 1m \times (L/2-d) + (P \times b)/L$$

where l = length of the clear span and the other terms are explained in Appendix C. V is determined to be 120 kN. The shear resistance is given by

$$V_r = 0.2 \times \phi \times \sqrt{f'_c} \times b \times d$$

Vr is determined to be 140 kN. Therefore, the slab has sufficient shear strength.

## 6.0 CONCLUSIONS AND RECOMMENDATIONS

1. A round timber, pile wharf is an acceptable alternative to crib wharves in sheltered harbours, where bottom conditions permit the driving of wooden piles.

2. This type of wharf costs less than any of the other types currrently in use, and is especially cost effective in deeper water.

3. An 8 inch concrete deck is recommended for the wharves. To accommodate this the piles should be placed to form a 3m x 3m section between pile caps.

4. On site inspection of the wharf construction should be carried out and particular attention should be paid to the placing and curing of the concrete deck.

5. It is recommended that a thorough engineering study be completed to determine the best design for the concrete deck to ensure adequate support for the expected loadings and to avoid over design of the structure.

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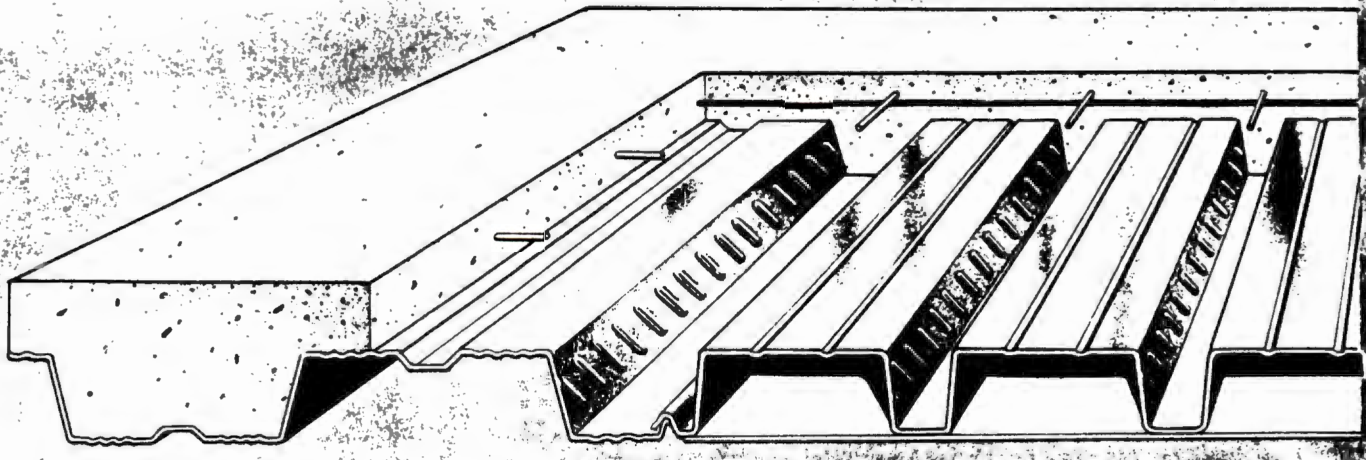
Westeel-Rosco Limited, Hi-bond Steel Floor Systems Design Manual

APPENDIX A.

excerpts from Westeel-Rosco Design Manual

WESTEEL-ROSCO LIMITED 04/77 5p METAL DECKING / STEEL FLOOR

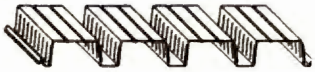


# HI-BOND®
# STEEL FLOOR SYSTEMS
# DESIGN MANUAL

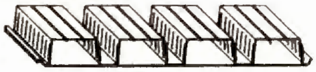
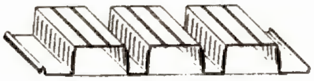
CSSBI
MEMBER

**WESTEEL-ROSCO LIMITED**

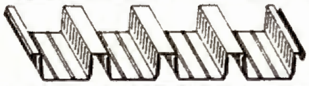
# HI-BOND PROFILE SELECTION CHART

| Type | Profiles | Unit Width (inches) | Rib Centres (inches) | Cell Area (inches)$^2$ | Concrete Usage* Cu. Yd./Sq. | Blend Profiles |
|---|---|---|---|---|---|---|
| T-168 |  | 40.00" std. | 8.00 | – | .915 | T-168F |
| T-15 |  | 24.00" std. Available in 18.00" | 6.00 | – | .946 | T-15F<br>T-168F |
| T-30-8 |  | 32.00" std. Available in 24.00" | 8.00 | – | 1.042 | T-30-8F<br>T-30-6F<br>T-30-8Fs<br>T-30-6Fs |
| T-30-6 |  | 24.00" std. Available in 18.00" | 6.00 | – | 1.133 | T-30-8F<br>T-30-6F<br>T-30-8Fs<br>T-30-6Fs |
| T-30 V |  | 32.00" std. Available in 30.00" | 16.00 | – | 1.192 | T-30-8F<br>T-30-6F<br>T-30-8Fs<br>T-30-6Fs |

## CELLULAR PROFILES

| | | | | | | |
|---|---|---|---|---|---|---|
| T-168 F |  | 40.00" std. | 8.00 | 9.710 | .915 | T-168<br>T-15 |
| T-15 F |  | 24.00" std. Available in 18.00" | 6.00 | 5.60 | .946 | T-15 |
| T-30-8 F |  | 32.00" std. Available in 24.00" | 8.00 | 16.98 | 1.042 | T-30-8<br>T-30-6<br>T-30V |
| T-30-8 $F_s$ |  | 28.00" std. | 8.00 | 16.98 | 1.134 | T-30-8<br>T-30-6<br>T-30V |
| T-30-6 F |  | 24.00" std Available in 18.00" | 6.00 | 10.98 | 1.133 | T-30-8<br>T-30-6<br>T-30V |
| T-30-6 $F_s$ |  | 28.00" std. | 6.00 | 10.98 | 1.212 | T-30-8<br>T-30-6<br>T-30V |

## INVERTED PROFILES

| | | | | | | |
|---|---|---|---|---|---|---|
| T-15 INV |  | 24.00" std. Available in 18.00" | 6.00 | – | 1.060 | – |
| T-30-8 INV |  | 32.00" std. Available in 24.00" | 8.00 | – | 1.427 | – |
| T-168 INV |  | 40.00" std. | 8.00 | – | 1.146 | – |

*Concrete usages are based on 2.50" concrete topping. For other thicknesses see technical data sheets.

# PROFILE SECTION PROPERTIES (FOR COMPOSITE PROPERTIES SEE TECHNICAL DATA SHEETS)

| Core Nominal Thickness (inches) | Weight (p.s.f.) | As (in.$^2$/ft.) | Sm (in$^3$/ft.) | Ss (in$^3$/ft.) | Sb (in$^3$/ft.) | Ie (in$^4$/ft.) | If (in$^4$/ft.) | YB (in) | Po (lbs) | PI (lbs) |
|---|---|---|---|---|---|---|---|---|---|---|
| .030 | 1.688 | .470 | .172 | .186 | .195 | .163 | .221 | 1.133 | 358 | 792 |
| .036 | 2.025 | .563 | .211 | .233 | .233 | .212 | .265 | 1.136 | 576 | 1174 |
| .048 | 2.688 | .751 | .291 | .308 | .308 | .324 | .352 | 1.144 | 1097 | 2142 |
| .060 | 3.360 | .938 | .372 | .382 | .382 | .440 | .440 | 1.151 | 1716 | 3377 |
| .030 | 1.799 | .495 | .206 | .203 | .234 | .178 | .203 | .870 | 486 | 1072 |
| .036 | 2.159 | .594 | .267 | .270 | .279 | .232 | .244 | .873 | 767 | 1582 |
| .048 | 2.867 | .792 | .366 | .368 | .368 | .325 | .325 | .881 | 1434 | 2877 |
| .060 | 3.561 | .989 | .456 | .456 | .456 | .405 | .405 | .889 | 2232 | 4534 |
| .030 | 2.161 | .599 | .361 | .411 | .455 | .609 | .857 | 1.885 | 415 | 852 |
| .036 | 2.589 | .718 | .470 | .535 | .544 | .785 | 1.028 | 1.890 | 709 | 1281 |
| .048 | 3.452 | .957 | .659 | .720 | .720 | 1.186 | 1.367 | 1.898 | 1412 | 2352 |
| .060 | 4.315 | 1.195 | .849 | .894 | .894 | 1.646 | 1.705 | 1.907 | 2237 | 3702 |
| .030 | 2.460 | .678 | .481 | .516 | .571 | .812 | .963 | 1.686 | 554 | 1136 |
| .036 | 2.946 | .813 | .626 | .672 | .683 | 1.046 | 1.155 | 1.690 | 946 | 1708 |
| .048 | 3.928 | 1.084 | .878 | .905 | .905 | 1.537 | 1.537 | 1.699 | 1883 | 3136 |
| .060 | 4.910 | 1.354 | 1.123 | 1.123 | 1.123 | 1.917 | 1.917 | 1.708 | 2983 | 4937 |
| .030 | 1.715 | .477 | .327 | .397 | .463 | .622 | .757 | 1.636 | 310 | 620 |
| .036 | 2.058 | .572 | .468 | .526 | .554 | .823 | .909 | 1.639 | 525 | 1050 |
| .048 | 2.712 | .762 | .726 | .736 | .736 | 1.211 | 1.211 | 1.646 | 1000 | 2000 |
| .060 | 3.390 | .953 | .916 | .916 | .916 | 1.514 | 1.514 | 1.653 | 1530 | 3070 |

T-30-

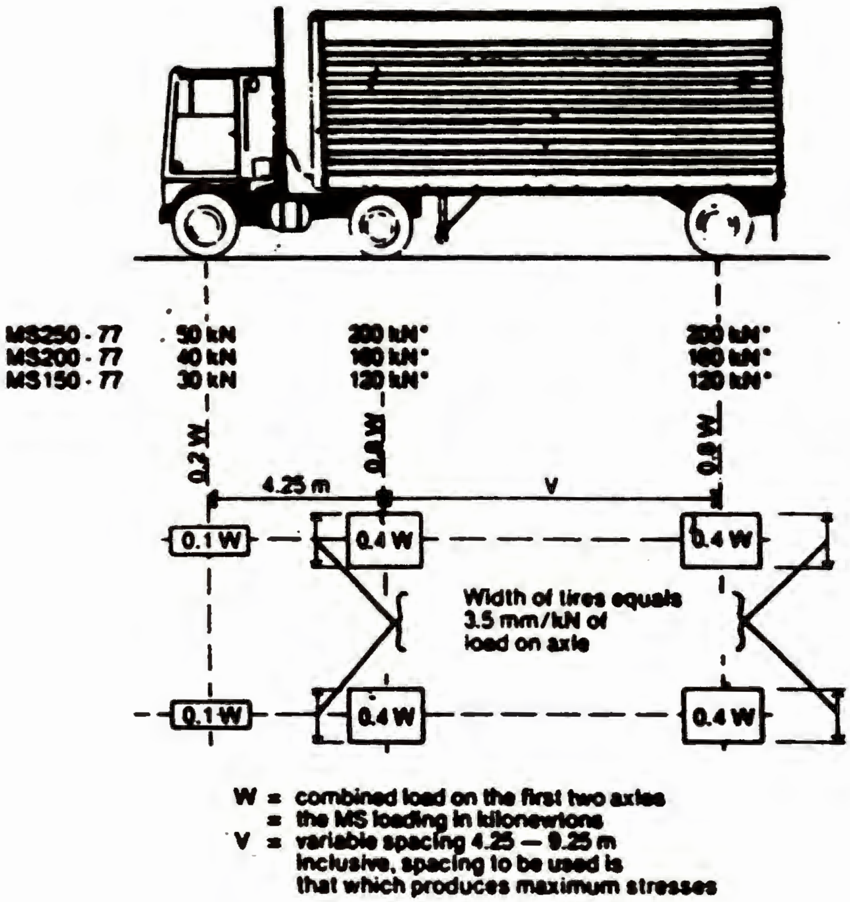
T-30-(

| Core Nominal Thickness (inches) | Weight (p.s.f.) | As (in.$^2$/ft.) | Sm (in$^3$/ft.) | Ss (in$^3$/ft.) | Sb (in$^3$/ft.) | Ie (in$^4$/ft.) | If (in$^4$/ft.) | YB (in) | Po (lbs) | PI (lbs) |
|---|---|---|---|---|---|---|---|---|---|---|
| .036/.036 | 3.516 | .995 | .275 | .385 | .916 | .398 | .582 | .635 | 576 | 1174 |
| .036/.048 | 4.013 | 1.139 | .280 | .492 | 1.156 | .429 | .635 | .550 | 576 | 1174 |
| .048/.048 | 4.676 | 1.327 | .435 | .582 | 1.227 | .618 | .781 | .637 | 1097 | 2142 |
| .048/.060 | 5.172 | 1.471 | .442 | .659 | 1.471 | .659 | .837 | .569 | 1097 | 2142 |
| .060/.060 | 5.844 | 1.658 | .630 | .801 | 1.541 | .874 | .983 | .638 | 1716 | 3377 |
| .036/.036 | 3.626 | 1.026 | .318 | .385 | .875 | .369 | .436 | .498 | 767 | 1582 |
| .036/.048 | 4.150 | 1.170 | .324 | .400 | 1.088 | .427 | .469 | .432 | 767 | 1582 |
| .048/.048 | 4.858 | 1.368 | .505 | .518 | 1.172 | .598 | .586 | .500 | 1434 | 2877 |
| .048/.060 | 5.358 | 1.512 | .513 | .536 | 1.389 | .638 | .621 | .447 | 1434 | 2877 |
| .060/.060 | 6.052 | 1.709 | .715 | .655 | 1.471 | .757 | .738 | .502 | 2232 | 4534 |
| .036/.036 | 4.083 | 1.150 | .577 | .791 | 1.698 | 1.395 | 1.992 | 1.173 | 709 | 1281 |
| .036/.048 | 4.580 | 1.294 | .590 | .980 | 2.092 | 1.503 | 2.172 | 1.038 | 709 | 1281 |
| .048/.048 | 5.443 | 1.533 | .888 | 1.180 | 2.267 | 2.109 | 2.665 | 1.176 | 1412 | 2352 |
| .048/.060 | 5.941 | 1.677 | .904 | 1.319 | 2.664 | 2.246 | 2.852 | 1.070 | 1412 | 2352 |
| .060/.060 | 6.804 | 1.915 | 1.259 | 1.609 | 2.836 | 2.929 | 3.343 | 1.179 | 2237 | 3702 |
| .036/.036 | 3.712 | .986 | .495 | .678 | 1.456 | 1.196 | 1.708 | 1.173 | 608 | 1098 |
| .036/.048 | 4.131 | 1.109 | .506 | .840 | 1.793 | 1.288 | 1.861 | 1.038 | 608 | 1098 |
| .048/.048 | 4.949 | 1.314 | .762 | 1.011 | 1.943 | 1.808 | 2.285 | 1.176 | 1210 | 2016 |
| .048/.060 | 5.368 | 1.437 | .775 | 1.131 | 2.284 | 1.925 | 2.444 | 1.070 | 1210 | 2016 |
| .060/.060 | 6.186 | 1.642 | 1.079 | 1.380 | 2.431 | 2.510 | 2.865 | 1.179 | 1918 | 3173 |
| .036/.036 | 4.432 | 1.245 | .753 | .918 | 1.787 | 1.728 | 1.962 | 1.098 | 946 | 1708 |
| .036/.048 | 4.927 | 1.389 | .771 | .961 | 2.165 | 1.866 | 2.120 | .979 | 946 | 1708 |
| .048/.048 | 5.909 | 1.660 | 1.160 | 1.244 | 2.384 | 2.601 | 2.624 | 1.101 | 1883 | 3136 |
| .048/.060 | 6.404 | 1.804 | 1.182 | 1.296 | 2.765 | 2.773 | 2.789 | 1.009 | 1883 | 3136 |
| .060/.060 | 7.386 | 2.074 | 1.643 | 1.580 | 2.980 | 3.336 | 3.291 | 1.104 | 2983 | 4937 |
| .036/.036 | 4.015 | 1.068 | .646 | .787 | 1.532 | 1.482 | 1.682 | 1.098 | 811 | 1464 |
| .036/.048 | 4.434 | 1.191 | .661 | .824 | 1.855 | 1.599 | 1.817 | .979 | 811 | 1464 |
| .048/.048 | 5.353 | 1.423 | .994 | 1.067 | 2.043 | 2.229 | 2.249 | 1.101 | 1614 | 2688 |
| .048/.060 | 5.772 | 1.546 | 1.013 | 1.111 | 2.370 | 2.377 | 2.390 | 1.009 | 1614 | 2688 |
| .060/.060 | 6.273 | 1.778 | 1.409 | 1.354 | 2.555 | 2.860 | 2.821 | 1.104 | 2557 | 4231 |

| Core Nominal Thickness (inches) | Weight (p.s.f.) | As (in.$^2$/ft.) | Sm (in$^3$/ft.) | Ss (in$^3$/ft.) | Sb (in$^3$/ft.) | Ie (in$^4$/ft.) | If (in$^4$/ft.) | YB (in) | Po (lbs) | PI (lbs) |
|---|---|---|---|---|---|---|---|---|---|---|
| .030 | 1.799 | .495 | .203 | .206 | .308 | .203 | .203 | .660 | 486 | 1072 |
| .036 | 2.159 | .594 | .270 | .267 | .368 | .244 | .244 | .663 | 767 | 1582 |
| .048 | 2.867 | .792 | .368 | .366 | .487 | .325 | .325 | .667 | 1434 | 2877 |
| .060 | 3.561 | .989 | .456 | .456 | .603 | .405 | .405 | .671 | 2232 | 4534 |
| .030 | 2.161 | .599 | .411 | .361 | .749 | .858 | .857 | 1.145 | 415 | 852 |
| .036 | 2.589 | .718 | .535 | .470 | .897 | 1.028 | 1.028 | 1.146 | 709 | 1281 |
| .048 | 3.452 | .957 | .720 | .659 | 1.189 | 1.367 | 1.367 | 1.150 | 1412 | 2352 |
| .060 | 4.315 | 1.195 | .894 | .849 | 1.479 | 1.705 | 1.705 | 1.153 | 2237 | 3702 |
| .030 | 1.688 | .470 | .186 | .172 | .383 | .221 | .221 | .577 | 358 | 792 |
| .036 | 2.025 | .563 | .233 | .211 | .457 | .265 | .265 | .580 | 576 | 1174 |
| .048 | 2.688 | .751 | .308 | .291 | .603 | .353 | .353 | .584 | 1097 | 2142 |
| .060 | 3.360 | .938 | .382 | .372 | .746 | .440 | .440 | .589 | 1716 | 3377 |

# APPENDIX B.

## Vehicle Loadings




MS LOADING CONFIGURATION

Source: CSA CAN3-S6-M78

# APPENDIX C.

## Detailed Calculations

# FORMWORK DESIGN

Method: A specific type of formwork is selected whose properties are known. This formwork is then checked to determine whether or not it is sufficent to handle expected loadings. The span length and concrete weight are known, and a slab thickness is assumed. The steel form must be able to withstand forces applied during pouring of the concrete.

Deflection of the steel must be within accepted limits, and the stress applied must be less than the nominal strength of the steel.

CALCULATIONS: CHOOSE WESTEEL - ROSCO LIMITED

HI-BOND T-30-6 from selection chart.

ASSUME SLAB THICKNESS OF 200 mm

## DEFLECTION

deflection, $\Delta = \dfrac{5}{384}\dfrac{WL^4}{EI}$ | CISC P. 5-9(

where $W$ = load per meter due to wet concrete

$L$ = span length, 3m

$E$ = Modulus of Elasticity, 200 GPa

$I$ = Moment of Inertia, $2610.6 \times 10^3 mm^4$

WESTEEL-ROSCO p. 3

$$W = 2400 \frac{kg}{m^3} \times 9.81 \frac{m}{s^2} \times 1m \times \frac{200}{1000} = 4.7 \frac{kN}{m}$$

assuming slab thickness = 200mm

and concrete density = $2400 kg/m^3$

$$\Delta = \frac{5}{384} \frac{(4.7 \times 10^3 kN/m^3)(3m)^4}{200 \times 10^9 \frac{N}{m^2} \times 2610.6 \times 10^3 mm^4} = 9.5 mm$$

$$9.5mm < L/180 = \frac{3000 mm}{180} = 16.7 \quad OK$$

Westeel ROSCO P. 6

$\therefore$ The selected material meets the requirement for deflection.

## STRESS IN STEEL

Stress is calculated using the applied moment, M and the section modulus, S.

$$STRESS = \frac{M}{S}$$

$$M = \frac{WL^2}{8}$$ where $W = 4.7 \times 10^3 \frac{KN}{m}$ previously calculated

$L$ = length of span

CPCA P. 1-41

$$M = \frac{4.7 \times 10^3 \frac{KN}{m} \times (3m)^2}{8} = 5.3 \ KN \cdot m$$

due to wet concrete

$$S = 60.18 \times 10^3 mm^3$$

WESTEEL - ROSCO P. 3

$$STRESS = \frac{5.3 \ KN \cdot m}{60.18 \times 10^3 mm^3} = 8.8 \times 10^7 \ KN/m^2$$

$$8.8 \times 10^7 < F_y/1.6 = \frac{227 \times 10^6}{1.6} = 14.1 \times 10^7 \frac{KN}{m^2}$$ OK

WESTEEL - ROSCO P. 6

∴ The selected material meets stress requirements.

# SLAB DESIGN

Method:

The concrete slab must be designed to withstand the following loads.

Dead load, consisting of the self weight of the slab and the equipment that may be permanently placed on the wharf.

Live Load, consisting of vehicle weight plus impact fraction, and snow loads.

The moment and shear is determined for maximum load combination and the slab is designed to withstand the loads. The amount of reinforcement used controls the maximum resistant moment while the thickness of the slab determines shear resistance.

CSA Design for Highway Bridges will be used as a design aid because a wharf deck subjected to traffic loads responds in a similar manner to a loaded highway bridge.

## Dead Load

The load due to the slab weight has previously been determined as 4.7 kPa.

The Department of Public Works recommends a deck load of about 19 kPa (400 lb/ft$^2$) to account for permanent infrastructure. | Kenny p. 10

The total dead load is 4.7 kPa + 19.1 kPa = 23.8 kPa

The total factored dead load is 1.25 (23.8 kPa)

= 30 kPa

## Live Load

The live load due to snow is calculated by the equation $S = C_s S_o$ | NBC supplement p. 196

where $C_s$ = snow load coefficient

$S_o$ = ground snow load

$C_s$ can be taken as 0.6 where wind conditions are prevalent. The ground snow load is taken as the highest for the island of Newfoundland which is 5.7 KN/m$^2$ at St. Anthony. | NBC Supplement p. 21

Therefore,

$$S = 0.6 \times 5.7 \frac{kN}{m^2} = 3.4 \frac{kN}{m^2}$$

The factored snow load becomes

$$1.5\,(3.4\ kPa) = 5\ kPa$$

The total uniform distributed load is equal to the total dead load + live load.

$$U.D.L. = 30\ kPa + 5\ kPa = 35\ kPa$$

For a wharf deck consisting of 4 equal spans and considering a one metre strip along the centre of the span the moment distribution is shown on the following page.

Hence, the largest positive moment occurs in the outside ~~span~~ panel.

$$\underset{(UDL)}{M_P} = 0.077 \times 35\ kPa \times (3\ m)^2 = 24\ kN \cdot m$$

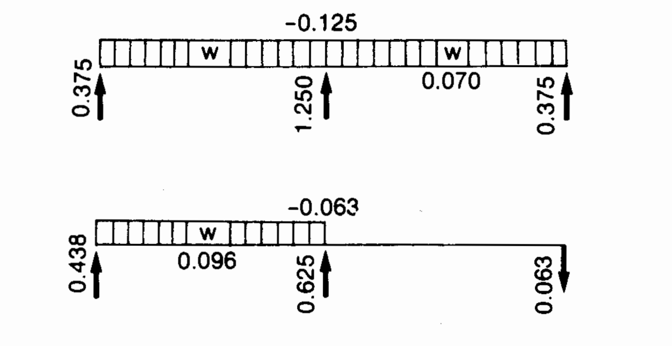
and the largest negative moment is

$$\underset{(UDL)}{M_N} = 0.107 \times 35\ kPa \times (3\ m)^2 = 34\ kN \cdot m$$

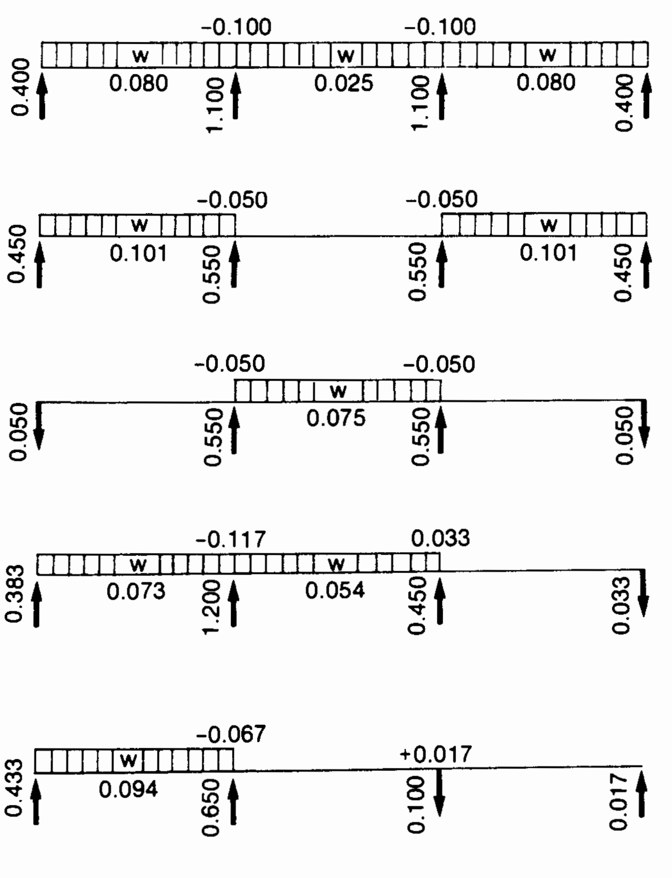
**Table 1.15: Moments and Reactions in Continuous Beams Uniformly Distributed Loads**

| | |
|---|---|
| **Moment:** Coefficient × $w\ell^2$<br>**Reaction:** Coefficient × $w\ell$ | **w:** Uniform load per unit length<br>**$\ell$:** Length of one span |

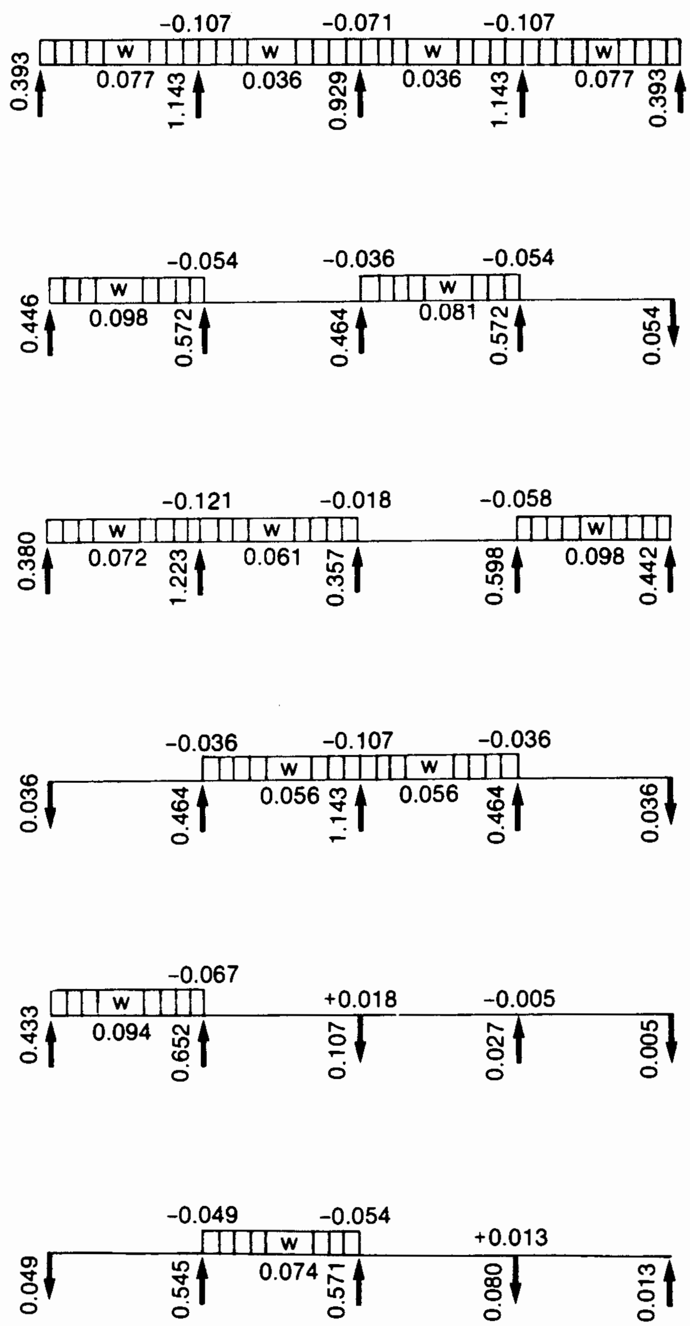
**Two Equal Spans**




**Three Equal Spans**




**Four Equal Spans**



## Vehicle Load

The vehicle loading is based on the wharf requirement for MS-200-77 loading, as shown in Appendix C.

Consider the wheel load as a point load at the centre of the span. This positioning produces the largest moment in the slab. | CAN/CSA-S6-88 5-2-8-1

Point load = $0.15(450\ \text{KN}) = 67.5\ \text{KN}$ | CAN/CSA-S6-88 5-3.3.2.2

This is the load applied by one wheel of an MS-200 loading.

The impact factor is 0.30 | CAN/CSA S6-88 5.2.11.3

Therefore, the total vehicle load becomes

$$(0.30+1.0)(67.5) = 88\ \text{KN}$$

This load is the design point load applied to the middle of the slab.

Moments

When the wharf is restricted to one MS-200 load the maximum moment occurs when the 88 kN load is applied to the centre of an outside span. See diagram on next page.

$$\underset{(P.L.)}{M_P} = 0.200 \times 88\,kN \times 3m = 53\ kN.m$$

$$\underset{(P.L)}{M_N} = 0.100 \times 88\,kN \times 3m = 26\ kN.m$$

The total positive moment is equal to the positive moment due to the U.D.L. and the positive moment due to the point load (P.L).

$$M_P = 24\,kN.m + 53\,kN.m = 77\,kN.m$$

and the total negative moment becomes

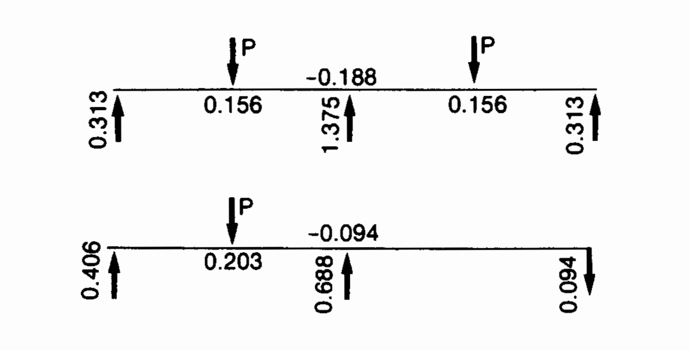
$$M_N = 34\,kN.m + 26\,kN.m = 60\,kN.m$$

These are the design moments for the longitudinal direction.

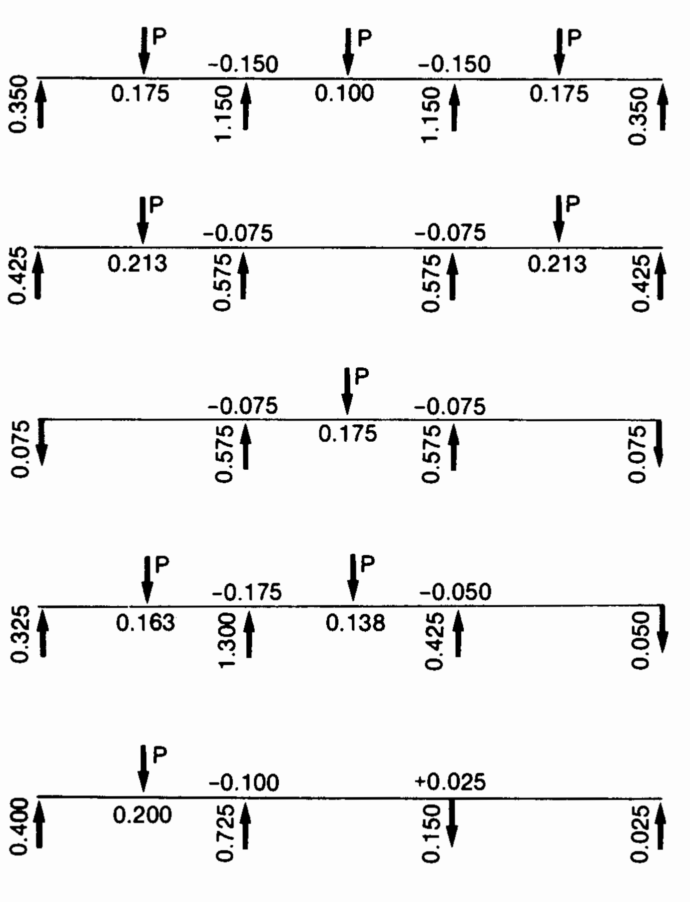
**Table 1.16: Moments and Reactions in Continuous Beams Central Point Loads**

| | |
|---|---|
| **Moment:** Coefficient × Pℓ<br>**Reaction:** Coefficient × P | **P:** Concentrated load<br>**ℓ:** Length of one span |

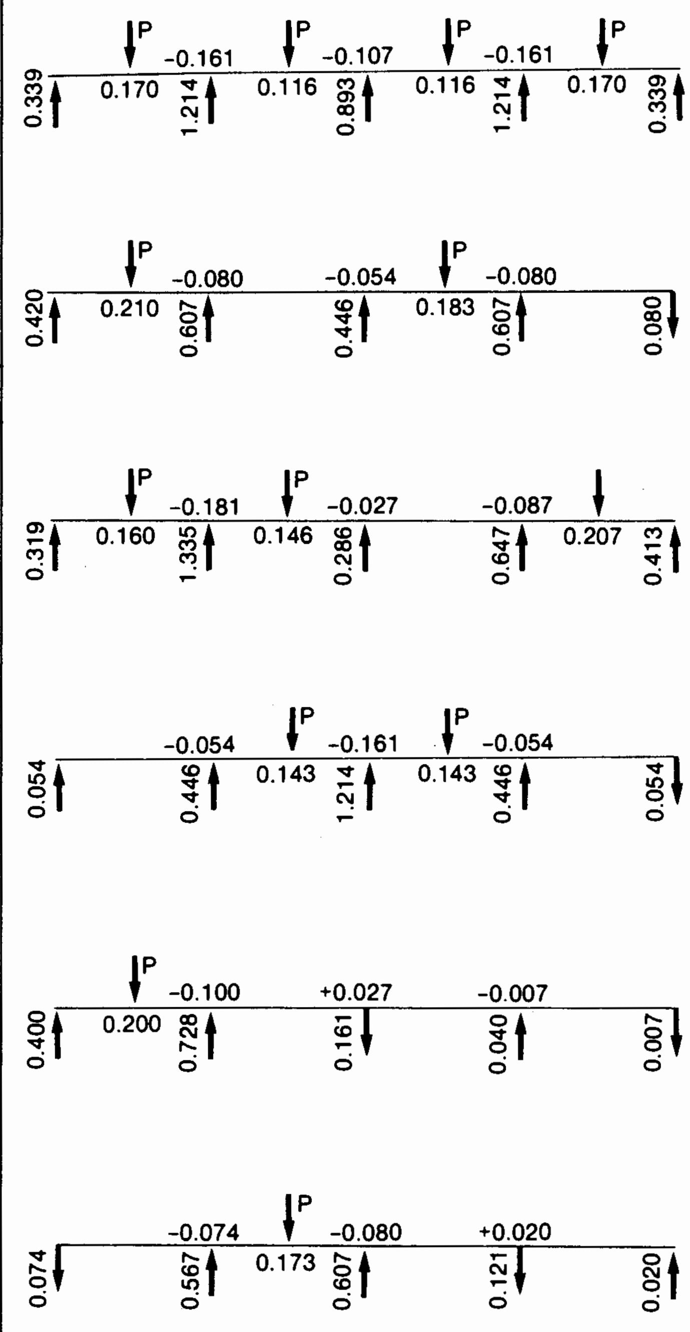
**Two Equal Spans**




**Three Equal Spans**




**Four Equal Spans**



The moments have been calculated by assuming that the slab panels have one-way bending action. While this is not exactly the case, the procedure is simplified by making this assumption. The coefficient method and the direct design method of two-way slab analysis is not applicable in this case. Extensive detailed analysis, such as equivalent frame analysis, is necessary for a complete and accurate analysis of the wharf deck. Also, a check of the analysis using a software package would be helpful.

For the purpose of calculation only it is assumed that the action is one way. The amount of transverse reinforcement provided will be equal to the amount of main reinforcement. Also, the amount of positive reinforcement will be the same in all panels, and the amount of negative reinforcement will be the same for all panels. In both cases it will be equal to the amount necessary for the panels with the greatest moment. This makes it easy to put the rebar in place because it becomes a grid pattern identical for each panel. However, it must be noted that this leads to a conservative design and the amount of reinforcing steel used could probably be reduced if a more accurate analysis was done.

## Slab thickness

$$h = \frac{S+3000}{30}$$

CAN/CSA-S6-88 Table 29

where $h$ = Slab thickness

$S$ = distance between centre of spans in millimetres

CAN/CSA-S6-88 5.3.3.1

$$h = \frac{3000+3000}{30} = 200\,mm$$

the minimum cover for reinforcing for a slab is 50mm, but this can be reduced to 40mm because of the protection of the steel formwork.

CAN/CSA-S6-88 Table 27

therefore, $d = 200mm - 40mm - \frac{1}{2}(20mm) = 150\,mm$

where 20mm is diameter of bar.

## reinforcement

calculate reinforcement per 1m strip

$$K_r = \frac{M \times 10^6}{bd^2} = \frac{77 \times 10^6}{(1000)(150)^2} = 3.42$$

CPCA table 2.3

$\rho = 1.16\% = 0.0116$ from table 2.3 CPCA
assuming concrete strength = 30 MPa

Area of steel required: $A_s = \rho b d$

$A_s = 0.0116 \times 1000 \times 150 = 1740 \text{ mm}^2/\text{m}$

minimum allowable $A_s = 0.002 \times$ concrete area

$A_s = 0.002 \times 1000 \times 200 = 400 \text{ mm}^2/\text{m}$

therefore, the amount of steel used $= 1740 \text{ mm}^2/\text{m}$

1, #20 bar $= 300 \text{ mm}^2$ | RIZKALLA & THADANI p. 15

# of bars $= \dfrac{1740}{300} = 5.8 = 6$ bars

Check minimum & maximum allowable spacing

**MINIMUM** NOT LESS THAN 40 mm | CAN/CSA-S6-88 8.2.2.1

**MAXIMUM** NOT MORE THAN $1.5 \times 200 = 300$ mm | 8.2.2.7

SPACING $= \dfrac{1000}{6} = 167$ mm

$40 < 167 < 300$ o.k.

use 6 #20M bars at 165 mm spacing

For negative reinforcement use 50 mm for cover since no protection is afforded by the formwork following previous procedure:

$$d = 200 - 50 - \frac{1}{2}(20) = 140\,mm$$

$$K_r = \frac{60 \times 10^6}{1000 \times 140^2} = 3.06$$

$$\rho = 1.01\% = 0.0101$$

$$A_s = 0.0101 \times 1000 \times 140 = 1414\,mm^2/m$$

5 #20 bars can be used. However, for simplification 6 #20 bars should be used so the pattern is identical to the positive reinforcement.

Shear check:

The shear force, $V_f$, in the slab can be approximated by considering a one metre strip of the slab and determining the shear due to the uniformly distributed load and the point load. Assume the maximum shear force occurs at a distance d from the beam support, and the point load due to the vehicle is located at the same distance d from the support.

$$V_f = W_f \times 1m \times \left(\frac{l_n}{2} - d\right) + \frac{Pb}{l_n}$$

where $l_n$ = length of clear span and the other terms are explained by the diagrams on the following pages.

$$V_f = 35 kN \times 1m \times \left(\frac{2.7m}{2} - 0.15m\right) + \frac{88 kN \times 2.4m}{2.7m}$$ CPCA Table 1-14

$$V_f = 120 kN$$

The resistant shear of the slab is given by — KIRK & PILLAI P. 177

$$V_c = 0.2 \phi \lambda \sqrt{f'_c}\, b d$$

$$V_c = 0.2(0.85)(1.0)\sqrt{30} \times 1000 \times 150 = 140 kN$$

Therefore, the slab has sufficient shear strength.

**Table 1.14: Beam Diagrams**

**Simple Beam — uniformly distributed load**

$R = V$ .................................... $= \frac{w\ell}{2}$

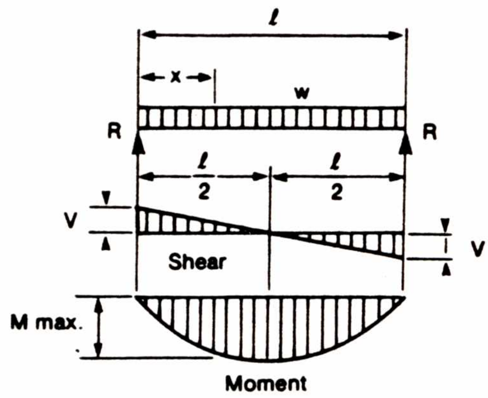
$V_x$ .................................... $= w\left(\frac{\ell}{2} - x\right)$

M max. (at centre) ........................ $= \frac{w\ell^2}{8}$

$M_x$ .................................... $= \frac{wx}{2}(\ell - x)$

$\Delta$max. (at centre) ........................ $= \frac{5\,w\ell^4}{384\,EI}$

$\Delta_x$ .................................... $= \frac{wx}{24\,EI}(\ell^3 - 2\ell x^2 + x^3)$




**Simple Beam — load increasing uniformly to one end**

$R_1 = V_1$ .................................... $= \frac{w\ell}{6}$

$R_2 = V_2$ .................................... $= \frac{w\ell}{3}$

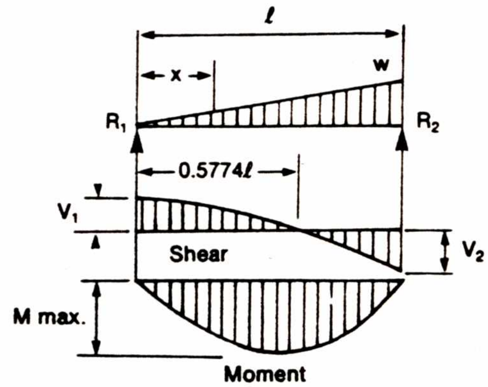
$V_x$ .................................... $= \frac{w\ell}{3} - \frac{wx^2}{2\ell}$

M max. $\left(\text{at } x = \frac{\ell}{\sqrt{3}} = 0.5774\ell\right)$ .......... $= \frac{w\ell^2}{9\sqrt{3}} = 0.06415\ell^2$

$M_x$ .................................... $= \frac{wx}{6\ell}(\ell^2 - x^2)$

$\Delta$max. $\left(\text{at } x = \ell\sqrt{1 - \sqrt{\frac{8}{15}}} = 0.5193\ell\right)$ ...... $0.00652\frac{w\ell^4}{EI}$

$\Delta_x$ .................................... $= \frac{wx}{360\,EI\ell}(3x^4 - 10\ell^2x^2 + 7\ell^4)$




**Simple Beam — load increasing uniformly to centre**

$R = V$ .................................... $= \frac{w\ell}{4}$

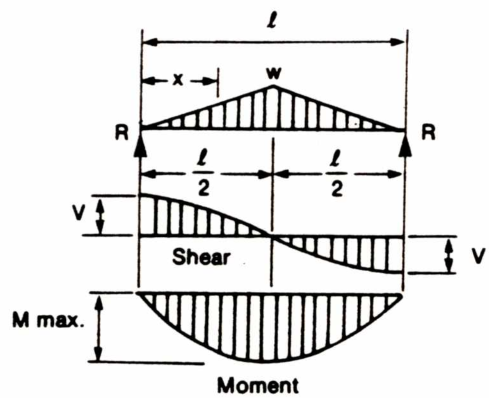
$V_x \left(\text{when } x < \frac{\ell}{2}\right)$ ........................ $= \frac{w}{4\ell}(\ell^2 - 4x^2)$

M max. (at centre) ........................ $= \frac{w\ell^2}{12}$

$M_x \left(\text{when } x < \frac{\ell}{2}\right)$ ........................ $= \frac{w\ell x}{2}\left(\frac{1}{2} - \frac{2x^2}{3\ell^2}\right)$

$\Delta$max. (at centre) ........................ $= \frac{w\ell^4}{120\,EI}$

$\Delta_x$ .................................... $= \frac{wx}{960\,EI\,\ell}(5\ell^2 - 4x^2)^2$




*Note:*
w: Distributed load per unit length. In the case of triangular distribution, w represents the maximum intensity of load per unit length.
P: Concentrated load.

## Table 1.14: (Continued)

**Simple Beam — uniform load partially distributed**

$R_1 = V_1$ (max. when $a < c$) ............... $= \frac{wb}{2\ell}(2c + b)$

$R_2 = V_2$ (max. when $a > c$) ............... $= \frac{wb}{2\ell}(2a + b)$

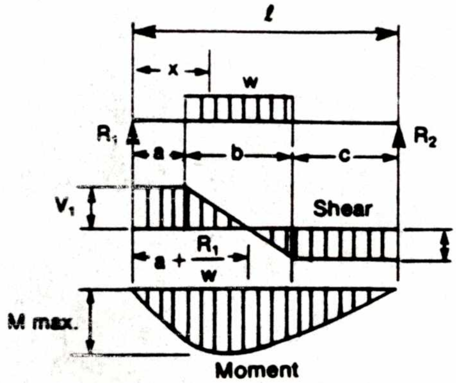
$V_x$ (when $x > a$ and $< (a + b)$) ............ $= R_1 - w(x - a)$

M max. $\left(\text{at } x = a + \frac{R_1}{w}\right)$ .................... $= R_1\left(a + \frac{R_1}{2w}\right)$

$M_x$ (when $x < a$) ......................... $= R_1 x$

$M_x$ (when $x > a$ and $< (a + b)$) ............ $= R_1 x - \frac{w}{2}(x - a)^2$

$M_x$ (when $x > (a + b)$) ..................... $= R_2(\ell - x)$




**Beam fixed at both ends — symmetrical trapezoidal load**

$R = V$ ........................................ $= \frac{w\ell}{2}\left(1 - \frac{a}{\ell}\right)$

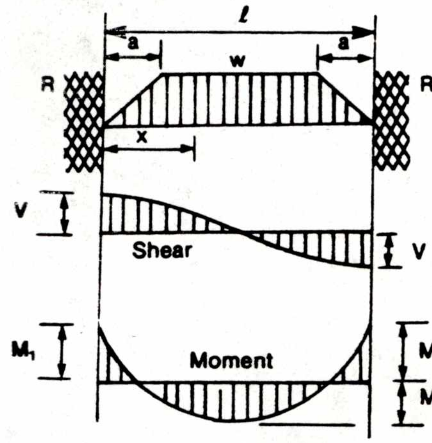
$M_1$ ............................................ $= \frac{w\ell^2}{12}\left(1 - 2\frac{a^2}{\ell^2} + \frac{a^3}{\ell^3}\right)$

$M_2$ ............................................ $= \frac{w\ell^2}{24}\left(1 - 2\frac{a^3}{\ell^3}\right)$

$M_x$ (when $x < a$) ............................ $= M_1 - R_x + \frac{wx^3}{6a}$

$M_x$ (when $a < x < \ell - a$) .................... $= M_1 - R_x + \frac{wa}{2}\left(x - \frac{2}{3}a\right) + \frac{w}{2}(x - a)^2$

Note: When $a = \ell/2$ loading is triangular




**Simple Beam — concentrated load at any point**

$R_1 = V_1$ (max. when $a < b$) ............... $= \frac{Pb}{\ell}$

$R_2 = V_2$ (max. when $a > b$) ............... $= \frac{Pa}{\ell}$

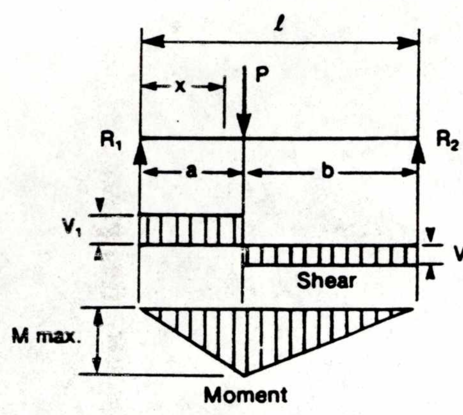
M max. (at point of load) ................... $= \frac{Pab}{\ell}$

$M_x$ (when $x < a$) ......................... $= \frac{Pbx}{\ell}$

$\Delta$max. $\left(\text{at } x = \sqrt{\frac{a(a + 2b)}{3}} \text{ when } a > b\right)$ $= \frac{Pab(a + 2b)\sqrt{3a(a + 2b)}}{27\,EI\ell}$

$\Delta a$ (at point of load) ...................... $= \frac{Pa^2b^2}{3\,EI\ell}$

$\Delta_x$ (when $x < a$) ......................... $= \frac{Pbx}{6\,EI\ell}(\ell^2 - b^2 - x^2)$




*Note:*

w: Distributed load per unit length. In the case of triangular distribution, w represents the maximum intensity of load per unit length.

P: Concentrated load.