

112 Verification of CSA Code
for Fixed Offshore
Concrete Structures

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VERIFICATION OF CSA CODE

FOR

FIXED OFFSHORE CONCRETE STRUCTURES

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The study team would also like to thank Gulf Canada and Mobil Oil Canada for supplying the design information for the Amauligak Caissons for the Beaufort Sea, and the Hibernia GBS structure for the Grand Banks of Newfoundland, respectively, which have contributed substantially to the success of this project.

EXECUTIVE SUMMARY

This study applied the provisions of two parts of the CSA Preliminary Standard for the Design, Construction, and Installation of Fixed Offshore Structures to the design of the reinforcing steel in the ice wall and immediately supporting walls of the Hibernia GBS 1986 Update Design. The two parts of the CSA Code that were applied are:

- Part I: S471-M1989, General Requirements, Design Criteria, the Environment and Loads.
- Part IV: S474-M1989, Concrete Structures.

A second design was developed using the following Norwegian codes:

- NPD REGULATIONS, Regulations for the Structural Design of Load Bearing Structures Intended for Exploitation of Petroleum Resources, issued by the Norwegian Petroleum Directorate, October 29, 1984. A revised, unofficial set of regulations, dated 1991, has been obtained for use with the most recent NS 3473 code.
- NORWEGIAN STANDARD (NS) 3473, Prosjektering ar Betongkonstruksjoner Beregnings - og Konstruksjonregler, 3rd Edition, 1989 (unofficial translation).

Previous Concrete Structures Verification Study

The previous verification study on concrete structures, G-2A, completed in November 1990 by Westmar Consultants Inc., performed design checks on the Amauligak production structure in the Beaufort Sea. In that study, the CSA code was verified, principally against the DnV code.

The loads developed in G-2A for the Hibernia GBS structure on the Grand Banks of Newfoundland were used herein.

Hibernia GBS Structure

The design supplied by Mobil Oil Canada for the Hibernia GBS structure was the 1986 preliminary update design. The information provided for the G-2B verification consisted of the overall characteristics and the concrete outline. No reinforcement or prestressing information was provided. The concept has a 1.4 m thick ice wall with 30 gear teeth and a diameter of 104 m from tip to tip of the teeth. The structure is for oil production in 80 m of water, with an operating life of 18 years. The ice wall is supported by a 0.55 m thick tie wall and 0.7 m thick support walls.

SHELL474

The computer program SHELL474 was developed in 1989 as CSA Verification Project E-2 to calculate, in accordance with CSA S474, the factored sectional resistances of a reinforced concrete element from given concrete and reinforcement dimensions, material grades and loading ratios and magnitudes.

SHELL474 has been upgraded to correct minor program bugs, to make it more user-friendly, to calculate post peak response, to calculate more realistic reinforcement stresses at cracks, and to print out detailed summaries of stresses and strains of each ultimate limit state (ULS) load stage to allow determination of the failure mode.

These upgrades to SHELL474 have enabled the efficient execution of the tasks required to carry out Verification Project G-2B.

Results of Design

The weights of steel in the central portions of the walls, away from the congested nodes and tips of the gear teeth in the design, are given in Section 6.0 of this report.

Detailing accounts for minimum reinforcement requirements and the practicalities of construction. The minimum reinforcement requirements are:

CSA-S474

As given in Clause 10.2.1 of S474, the area of reinforcement near each face and in each of the two orthogonal reinforcement directions shall not be less than .003 times the area of the concrete section, nor shall it be less than the area calculated using the following equation:

$$A_s = (f_{cr} + w) b_{hcf}/f_y$$

NPD and NS 3473

As given in Section 7 of the concrete appendix to NPD (1991), the minimum reinforcement in walls and shells, on each side and in each direction is:

$$A_s = kA_c (f_{tk} + w)/f_{sk}$$

The symbols are defined in Section 4.8 of this report. The governing conditions for detailing are listed in Tables 6.1 to 6.6. of this report.

Appraisal of the CSA Code

The set of codes examined herein, namely CSA S471, and S474-M1989, demonstrate a high level of sophistication, particularly when applied to the ice laden waters of the Canadian Offshore.

The two computer codes developed in earlier verification projects, namely the DnV Probabilistic Framework for calculating global ice loads and SHELL474 for analyzing elements of the structure, add a level of sophistication to the design process that no other code writing bodies have attempted. The computer programs have contributed greatly to the execution of this project.

The areas of the code verified in this study are:

- Ultimate limit states (ULS) of strength.
- Serviceability limit states (SLS) of local damage and crack widths.
- Fatigue.
- Ductility.

The ULS analysis procedures were verified and found to work very well with the program SHELL474 providing for efficient design of the structural elements.

The SLS analysis procedures were also verified and found to work very well, with the exception that a new load combination number 10 was added to Table 6.2 of S471, for the purpose of this study, to account for a local damage control load for a rare environmental event, namely a 1 in 100 year iceberg; the return period of this more frequent, but rare environmental event, should also be re-examined as the ULS for the 1 in 100 year iceberg indicates a calculated load factor by SHELL474 of only 1.11 where 1.0 is the load factor in this load combination.

The SLS load of an annual wave could also be re-examined as it is conservative compared with NPD where a wave that is exceeded 100 times in the design life of the structure, about once every two months, is used.

The fatigue provisions in CSA were verified, and, for the case of the fatigue life of reinforcing steel acted on by waves over the 18 year life of the structure, were found to be conservative by 14% when compared with the approach used in NPD where:

- The stresses in the S-N curve are divided by 1.5 in the CSA approach.
- The number of cycles are multiplied by 10 in the NPD approach.

More guidance could, however, be provided in the commentary to S474 on the appropriate S-N curves to use; in this study, the ones put forward in NS 3473 were used.

The ductility provisions put forward in S474 were found to be excellent. To meet the required element ductility, it was calculated, using the Kent and Park stress-strain curve for confined concrete, that in the order of 4,000 mm²/m² of transverse steel is required. Ductility runs on SHELL474 show that there is insufficient ductility in the 7 support wall elements using the unconfined stress-strain curves in S474. It is proposed that the commentary to S474 provide some discussion on how to achieve the required ductility, such as by using confining steel as calculated using an appropriate confined stress-strain curve, such as the Kent and Park curve as utilized herein.

RÉSUMÉ

La présente étude a eu recours aux dispositions de deux parties préliminaires du code de la CSA portant sur la conception, la construction et l'installation de plates-formes marines fixes, pour la conception de l'acier d'armature du mur pare-glace et des murs de soutènement sous-jacents du modèle mis à jour de 1986 de la plate-forme à embase-poids Hibernia. Les deux parties du code de la CSA qui furent utilisées sont :

- Partie I : S471-M1989, General Requirements, Design Criteria, the Environment and Loads.
- Partie IV : S474-M1989, Concrete Structures.

Un second modèle fut élaboré à partir des codes norvégiens suivants :

- RÈGLEMENTS NPD, Règlements touchant la conception structurale de plates-formes porteuses destinées à l'exploitation des ressources pétrolières, publiés par l'organisme de réglementation de l'industrie pétrolière de la Norvège, le 29 octobre 1984. On s'est procuré un jeu non officiel révisé des règlements de 1991 afin de pouvoir les utiliser avec la norme NS 3473 la plus récente.

NORME NORVÉGIENNE NS 3473, Prosjektering ar Betongkonstruks- joner Beregnings - og Konstruksjonregler, 3^e édition, 1989 (traduction non officielle).

Étude de vérification antérieure des structures de béton

L'étude de vérification antérieure des structures de béton, G-2A, terminée en novembre 1990 par Westmar Consultants Inc., visait l'exécution de vérifications de calcul sur la plate-forme de production Amanligak dans la mer de Beaufort. Dans le cadre de cette étude, le code de la CSA a été vérifié surtout en regard du code DnV .

Les charges élaborées dans G-2A pour la plate-forme à embase-poids Hibernia sur les Grands Bancs de Terre-Neuve ont été utilisées dans le présent rapport.

Plate-forme à embase-poids Hibernia

Le modèle fourni par Mobil Oil Canada pour la plate-forme à embase-poids Hibernia fut le modèle préliminaire mis à jour de 1986. Les renseignements fournis pour la vérification G-2B consistaient en des caractéristiques globales et en un aperçu général du béton. Aucun renseignement n'était fourni sur l'armature ou la

précontrainte. Le concept prévoit un mur pare-glace de 1.4 m d'épaisseur constitué de 30 dents d'engrenage; le diamètre du mur, mesuré d'une pointe à l'autre des dents, est de 104 m. La plate-forme est destinée à la production de pétrole dans 80 m d'eau, et sa durée utile d'utilisation est de 18 ans. Le mur pare-glace est supporté par un mur de raccordement de 0.55 m d'épaisseur et des murs-supports de 0.7 m d'épaisseur.

SHELL474

Le programme informatique SHELL474 fut élaboré en 1989 dans le cadre du projet de vérification E-2 de la CSA afin de calculer, conformément à la norme CSA S474, les résistances sectionnelles pondérées d'un élément en béton armé à partir de dimensions d'armature et de béton, de catégories de matériau et de grandeurs et de rapports de charge donnés.

Le SHELL474 a été amélioré afin de corriger des erreurs mineures de programmation, de le rendre plus facile à utiliser, de pouvoir calculer la réaction post-pointe, de pouvoir calculer des contraintes d'armature plus réalistes aux fissures et de pouvoir obtenir sur imprimé d'ordinateur des résumés détaillés des contraintes et des déformations de chaque étape de charge à l'état-limite ultime (ULS), afin de déterminer le mode de défaillance.

Ces améliorations au SHELL474 ont permis l'exécution efficace des tâches requises pour réaliser le projet de vérification G-2B.

Résultats du modèle

Les poids de l'acier situé dans les parties centrales des murs, loin des noeuds encombrés et des pointes des dents d'engrenage dans le modèle, sont donnés à la Section 6.0 du présent rapport.

Rapports détaillés concernant les exigences minimales d'armature et les questions pratiques de construction. Les exigences minimales d'armature sont :

CSA-S474

Comme il est indiqué à l'article 10.2.1 de la norme CSA S474, la surface d'armature à proximité de chaque face et dans chacune des deux directions d'armatures orthogonales ne doit pas être inférieure à .003 fois la surface de la section de béton, ni ne doit être inférieure à la surface calculée d'après l'équation suivante :

$$A_s = (f_{cr} + W) bh_c / f_y$$

NPD et NS 3473

Comme l'indique la Section 7 de l'appendice portant sur le béton au NPD(1991), l'armature minimale dans les murs et les parois, de chaque côté et dans chaque direction, est :

$$A_s = kA_e (f_k + W)/f_{ak}$$

Les symboles sont définis à la Section 4.8 du présent rapport. Les conditions régissant les rapports détaillés sont énumérées dans les tableaux 6.1 à 6.6 du présent rapport.

Évaluation du code de la CSA

Les deux parties du code faisant l'objet de la présente étude, à savoir les normes CSA S471 et CSA S474-M1989, démontrent un niveau élevé de perfectionnement, surtout dans les applications concernant les eaux chargées de glace au large des côtes du Canada.

Les deux codes informatiques élaborés lors de projets antérieurs de vérification, à savoir le DnV Probabilistic Framework servant au calcul des charges globales de glace et le SHELL474, à l'analyse des éléments de la plate-forme, ajoutent au processus de calcul un niveau de sophistication auquel aucun autre organisme de codification ne s'est attaqué. Les programmes informatiques ont largement contribué à l'exécution de ce projet.

Les éléments du code qui sont vérifiés dans la présente étude sont :

- États-limites ultimes (ULS) de la résistance;
- États-limites de service (SLS) de largeurs de fissures et de dommages locaux;
- Fatigue;
- Ductilité.

Vérification faite, les méthodes d'analyse ULS ont démontré qu'elles travaillaient très bien avec le programme SHELL474 fournissant un calcul efficace des éléments structuraux.

Les méthodes d'analyse SLS ont elles aussi été vérifiées et ont aussi démontré qu'elles travaillaient très bien, sauf qu'il a fallu ajouter un nouveau nombre 10 de simultanéité des charges au Tableau 6.2 de la norme S471, aux fins de la présente étude, pour rendre compte d'une charge pour limiter les dommages locaux lors d'un fait environnemental rare, soit un (1) iceberg par 100 ans; la période de retour de ce fait environnemental plus fréquent, mais rare, devrait aussi être de nouveau étudié puisque que l'ULS pour un (1) iceberg par 100 ans indique un facteur de charge calculé par SHELL474 de seulement 1.11, où 1.0 est le facteur de charge dans cette simultanéité de charge.

La charge SLS d'une vague annuelle devrait aussi être étudiée de nouveau car elle est conservatrice en comparaison du NPD où est utilisée une vague qui est dépassée 100 fois lors de la vie utile de la plate-forme, environ une fois tous les deux mois.

Les dispositions sur la fatigue dans le code de la CSA ont été vérifiées et, en ce qui concerne la durée de vie en fatigue de l'acier d'armature occasionnée par les vagues pendant la durée d'utilisation (18 ans) de la plate-forme, se sont avérées conservatrices par 14 % comparativement à l'approche utilisée dans le NPD, où :

- Les contraintes dans la courbe S-N sont divisées par 1.5 selon l'approche de la CSA.
- Le nombre de cycles est multiplié par 10 selon l'approche du NPD.

En conséquence, le commentaire à la norme S474 pourrait contenir plus d'indications, à savoir quelles sont les courbes S-N appropriées qu'il faudrait utiliser; dans la présente étude, ce sont celles qui sont mises de l'avant dans la NS 3473 qui ont été utilisées.

Les dispositions sur la ductilité mises de l'avant dans la norme S474 se sont avérées être excellentes. Afin d'atteindre la ductilité exigée pour l'élément, il a été calculé, à l'aide de la courbe contrainte-déformation Kent et Park pour du béton confiné, qu'il fallait environ 4000 mm²/m² d'armature transversale. La ductilité exécutée sur SHELL474 démontre que la ductilité est insuffisante dans les 7 éléments du mur-support en utilisant les courbes contrainte-déformation non confinées de la norme S474. Il est proposé que le commentaire à la norme S474 traite de la façon d'obtenir la ductilité requise, comme par l'utilisation d'acier de confinement, calculé en utilisant une courbe contrainte-déformation confinées appropriée, comme la courbe Kent et Park utilisée dans la présente.

1.0 INTRODUCTION

This project is the result of a Request for Proposals dated April 26, 1991. Westmar Consultants Inc. with associated subcontractors submitted a proposal on May 13, 1991 and was notified in writing by the CSA on May 27, 1991 of proposal acceptance and was given authorization to proceed. The project team assembled for this work includes:

- **Westmar Consultants Inc.**

Mr. N.F.B. Allyn, P.Eng.
Mr. A.J. Beattie, P.Eng.
Mr. S. Yee

- **Construction Technology Laboratories, Inc.**

Mr. W.J. Cichanski, P.E.
Mr. D.D. Magura, P.E.

- **Dr. P. Adebar, P.Eng., Assistant Professor, University of British Columbia**

The Technical Authority for Verification Project No. G-2B is Dr. M.P. Collins of the University of Toronto. The Scientific Authority is Dr. R.J. Smith of the National Energy Board (NEB). The project is funded through the Environmental Studies Research Funds (ESRF).

Verification Project G-2B applies the CSA Preliminary Standards S471 and S474 to a preliminary design of the gear tooth ice wall and immediate support members of the Hibernia GBS (1986 Update Design provided by Mobil Oil). A comparison is made with the requirements of the current Norwegian Petroleum Directorate (NPD) Regulations and the referenced Norwegian Standard (NS) 3473 in terms of required concrete dimensions and reinforcement details.

The body of this report summarizes:

- the upgrades made to SHELL474 to perform this study, including a copy of the latest version of the program;

- the Hibernia GBS test structure;

- the design criteria for the two codes for the principal loadings of icebergs and waves during the operation phase, and the related structural design considerations of strength, serviceability, ductility, fatigue and local damage;

- the finite element analysis carried out to determine the section forces; and,

- the design of the central portions of the ice, tie, and support walls away from the nodes (the effect of the detailing requirements on the reinforcement is also included in the designs) in accordance with the two codes.

The two designs are assessed and compared, with the reasons for the difference in the designs using the Canadian and Norwegian codes presented. Finally, the Canadian code is critically appraised, and specific areas are noted where the code committees should concentrate their efforts in preparing the codes for final issuance.

The two parts of the CSA Preliminary Standard for the Design, Construction, and Installation of Fixed Offshore Structures that are referenced and utilized herein are:

- Part I - S471-M1989, General Requirements, Design Criteria, the Environment and Loads.
- Part IV - S474-M1989, Concrete Structures

The Norwegian regulations and codes that are referenced and utilized herein are:

- NPD REGULATIONS, Regulations for the Structural Design of Load Bearing Structures Intended for Exploration of Petroleum Resources, issued by the Norwegian Petroleum Directorate, October 29, 1984. A revised, unofficial set of regulations, dated 1991, has been obtained for use with the most recent NS 3473 code.
- NORWEGIAN STANDARD (NS) 3473, Prosjektering av Betongkonstruksjoner Beregnings - og Konstruksjonsregler, 3rd Edition, 1989 (unofficial translation).

2.0 ENHANCEMENTS TO PROGRAM SHELL474

2.1 Background

Program SHELL474 was initially developed by Michael P. Collins, Perry Adebar and Uwe Kirschner as CSA Verification Project E-2 in April 1989 (Ref. 2.1). The intent of the project was to develop and document a computer program capable of calculating, in accordance with the requirements of CSA S474, the factored sectional resistances of a reinforced concrete element from given concrete dimensions, reinforcement ratios, material grades and loading ratios.

In 1990 program SHELL474 was used by Westmar Consultants Inc. to check various elements of the Amauligak production structure as part of Verification Project G-2A. After making numerous runs with program SHELL474, the design team at Westmar identified minor errors in the program. These errors were subsequently corrected by the original authors of SHELL474. In addition, the Westmar team suggested numerous enhancements to program SHELL474. Some of the more simple changes were implemented immediately by the authors of the program in consultation with the design team at Westmar.

As part of ongoing research into the behaviour of concrete offshore structures (Ref. 2.2 and 2.3), Perry Adebar and Michael P. Collins have made a number of further enhancements to program SHELL474. The most significant of these changes has been the addition of a specially developed user interface. This interface greatly increases the friendliness of SHELL474 since it allows the user to control the operation of the program and to input all data through "pull down menus." The user interface was written by Thomas Wong, a computer engineer in the Department of Civil Engineering, University of British Columbia. The latest version of SHELL474 is Version 4.10.

2.2 Summary of Enhancements Made to SHELL474

As part of the present verification study the following enhancements were made to program SHELL474.

1. Additional subroutines were developed so that the post-peak response of an element can be predicted. This enhancement was required in order to assess the system ductility requirements in CSA S474. In addition, as a result of this enhancement the new version of SHELL474 is less prone to instability while iterating for the peak load.
2. An additional subprogram was developed to print out a descriptive summary of the concrete strains and stresses as well as reinforcement stresses at each Ultimate Limit State load level. This enhancement allows the user to determine the failure mode of the element which is especially helpful when designing an element using program SHELL474.
3. The procedures used in program SHELL474 to check that the loads resisted by the average tensile stresses in the concrete can be transmitted across the crack have been modified. This allows more realistic estimates to be made of the reinforcement stresses.

4. SHELL474 has been modified so that the sectional forces can be either increased proportionally, kept constant or a combination of both.
5. An option has been added so that the program will automatically apply the material resistance factors required for system ductility check and fatigue check.
6. The maximum concrete compressive stresses at specified serviceability load levels are now printed out. This enhancement was required in order to assess the local damage requirements in CSA S474.

A more detailed description of the enhancements made to program SHELL474 is given in the following sections.

2.3 Predicting Post-Peak Response

The modification of SHELL474 so that it can predict the post-peak response of an element represents the most significant enhancement of SHELL474 to date. As mentioned previously this enhancement allows the system ductility requirements of CSA S474 to be assessed and also increases the overall reliability of program SHELL474.

The earlier versions of SHELL474 were based on what could be called a "load control" iterative scheme. That is, the program would iterate for a valid strain distribution (biaxial strains varying linearly over the depth plus three dimensional strains at the section mid-height) associated with a certain set of eight sectional forces (see Ref. 2.1). When a valid strain state was found the target vector would be increased proportionally by a certain increment. If a valid strain state could not be found the target load vector was reduced and the iteration was repeated with a smaller load increment. Once the load increment became smaller than a certain amount (1.5% of the initial specified load vector) it was assumed that the largest load vector was found and the iteration was stopped.

In the ULS system ductility mode of the new version of SHELL474, once the largest load vector has been found, the program switches automatically into a "strain controlled" iteration scheme. The program first determines which strain component is the largest. It then increments that strain and iterates for the remaining eight strain parameters which give the specified force ratios. That is, the program no longer iterates for a specific load vector but a load vector whose components have a certain proportion. Therefore as the strain is increased the magnitude of the load vector may increase or decrease and thus the program is sure to find the peak of the response as well as the post-peak response.

2.4 Subprogram DISPLAY

In order to get a descriptive summary of the concrete strains and stresses as well as the reinforcement stresses at each Ultimate Limit State load level an additional subprogram called DISPLAY was developed. Among other things, DISPLAY allows the user to determine the failure mode of an element.

After making a ULS analysis run with SHELL474 the user can examine the material stresses and strains at any load level by executing the STRESSES command from the main menu.

The user is then prompted by the screen:

Press

| | |
|-----|------------------------------|
| F1 | Inplane strains and stresses |
| F2 | Reinforcement stresses |
| F3 | 3-D strain and stresses |
| Esc | Quit |

If the user presses the F1 key, program DISPLAY summarizes the inplane strains and stresses at one particular layer of the element for one load stage. The default layer is the section mid-height of the element and the default load stage is the final load stage. By pressing the F1/F2 keys the user can increase/decrease the load stage number (ie. increase/decrease the load). With the F3/F4 keys the user can increase/decrease the elevation of the layer which is being examined.

If on the other hand, the user presses the F2 key while on the main menu a summary of the reinforcement stresses will be given for a particular load stage. Again, the F1/F2 keys can be used to change the load stage number.

Finally, if the user presses the F3 key when in the main menu a summary of the three dimensional strains and stresses at the section mid-height will be given. This information is useful when the failure mode is influenced by the transverse shear.

Note that the result file which contains a summary of the load stage data is still generated in the new version of SHELL474. This file can either be viewed from within the user interface by executing the VIEW command or the user can exit the program and use any screen editor to view the file. For a permanent record this file can of course be printed.

2.5 Transmitting Loads Across Cracks

CSA S474 states that tensile stresses in the concrete can be accounted for as long as it can be shown that the loads resisted by these tensile stresses can be transmitted across the cracks. In the initial version of SHELL474 a simple conservative approach was used for this check. A certain amount of the reinforcement capacity was reserved to transmit the forces across the cracks. This was done by calculating an "average yield stress" of the reinforcement which is lower than the yield stress of a bare bar. See Ref. 2.1 for a description of this approach.

In the new version of SHELL474 the average yield stress is assumed to be equal to the bare bar yield stress (ie. the yield stress is not reduced). Instead at each iteration point the program calculates how much capacity is still available in the reinforcement. If there is insufficient capacity in the reinforcing bars to transmit the forces across the cracks, the average tensile stress in the concrete is reduced. For example, if all the reinforcement has yielded the average tensile stress in the concrete is reduced to zero.

2.6 Specifying Constant Force Components

In the earlier versions of SHELL474 the eight sectional forces were assumed to be proportional. That is, in trying to find the maximum load factor the program would increase the specified sectional forces proportionally.

In the new version of SHELL474 the user can specify a proportional component as well as a constant component for each of the eight sectional forces. Note that all eight of the constant components can be zero, while at least one of the proportional components must be non-zero when making a ULS analysis run.

2.7 Automatic Material Resistance Factors

When doing an analysis using the new version of SHELL474 the user can specify whether it is a ULS sectional strength calculation, system ductility check, fatigue check, or an SLS analysis. The program then automatically applies the appropriate material resistance factors which are specified in CSA S474.

2.8 Maximum Concrete Compressive Stresses

In order to assess the local damage requirements in CSA S474, a small modification was made to SHELL474 so that during an SLS analysis it now calculates and prints out the maximum sectional concrete compressive stress.

2.9 References

1. Collins, Michael P.; Adebar, Perry; and Kirschner, Uwe, "SHELL474 - A Computer Program to Determine the Sectional Resistance of Concrete Offshore Structures in Accordance with CSA Standard S474-M89," *Canadian Standards Association Verification Project No. E-2 Report*, April 1989, 125 pp.
2. Adebar, Perry and Collins, Michael P., "A Consistent Shear Design Model for Concrete Offshore Structures," *IABSE Colloquium Report*, Vol. 62, Stuttgart, April 1991, pp. 457-462.
3. Adebar, Perry and Collins, Michael P., "Shear Design of Concrete Offshore Structures," Submitted to the American Concrete Institute for publication in the *Structural Journal*.

3.0 HIBERNIA GBS STRUCTURE

The test structure for the East Coast of Canada is the Hibernia GBS 1986 preliminary update design supplied to CSA by Mobil Oil Canada. This design was prepared by Newfoundland Offshore Development Constructors (NODECO) with the structure having a diameter of 104 m from tip to tip of the 30 gear teeth. The structure is for oil production in 80 m of water on the Grand Banks of Newfoundland.

The confidential information supplied to the project team by Mobil included:

- Drawings of the structure configuration including ice wall, internal walls and overall dimensions.
- Design criteria and forces.

The team has used the detailed information supplied by Mobil to build the finite element model of the structure.

The structure is shown in an elevation section in Figure 3.1, and in a plan view cross section in Figure 3.2.

The wall and slab element thicknesses are given in Table 3.1.

| Element | Thickness (m) |
|--------------------|-------------------------------|
| Base Slab | 3.80 |
| Top Slab | 0.60 |
| Ice Wall | 1.40 |
| Tie Wall | 0.55 |
| Drill Shaft Wall | 0.60 |
| Utility Shaft Wall | 1.10/0.60 ⁽¹⁾ |
| Riser Shaft Wall | 1.10 |
| Interior Wall | 0.70/0.60/0.50 ⁽²⁾ |

Table 3.1 Wall and Slab Thicknesses Used in the Finite Element Model

- Notes:
1. The thicker wall sections are used where full hydrostatic pressures can be applied to the shaft.
 2. Varying wall thicknesses are used depending on design pressure difference from oil, water, and solid ballast. Support walls, the interior walls immediately behind the ice wall, are 0.7 m thick.

Two kinds of concrete are used; namely, lightweight concrete for the base slab and normal weight concrete for all other elements. The properties of the concrete are given in Table 3.2.

| Concrete Type | Mass Density (kg/m ³) | Apparent Density (kg/m ³) | f _c (MPa) | E _c (MPa) | Poisson's Ratio ν | G (MPa) |
|------------------------|-----------------------------------|---------------------------------------|----------------------|----------------------|-------------------|---------|
| Lightweight | 1910 | 910 | 40 | 22701 | 0.30 | 8731 |
| Normal Weight Concrete | 2380 | 1380 | 50 | 35355 | 0.30 | 13598 |

Table 3.2 Properties of Concrete Density and Structure (Density and Strength are from Mobil, the remaining properties are calculated)

The design life of the structure is 18 years.

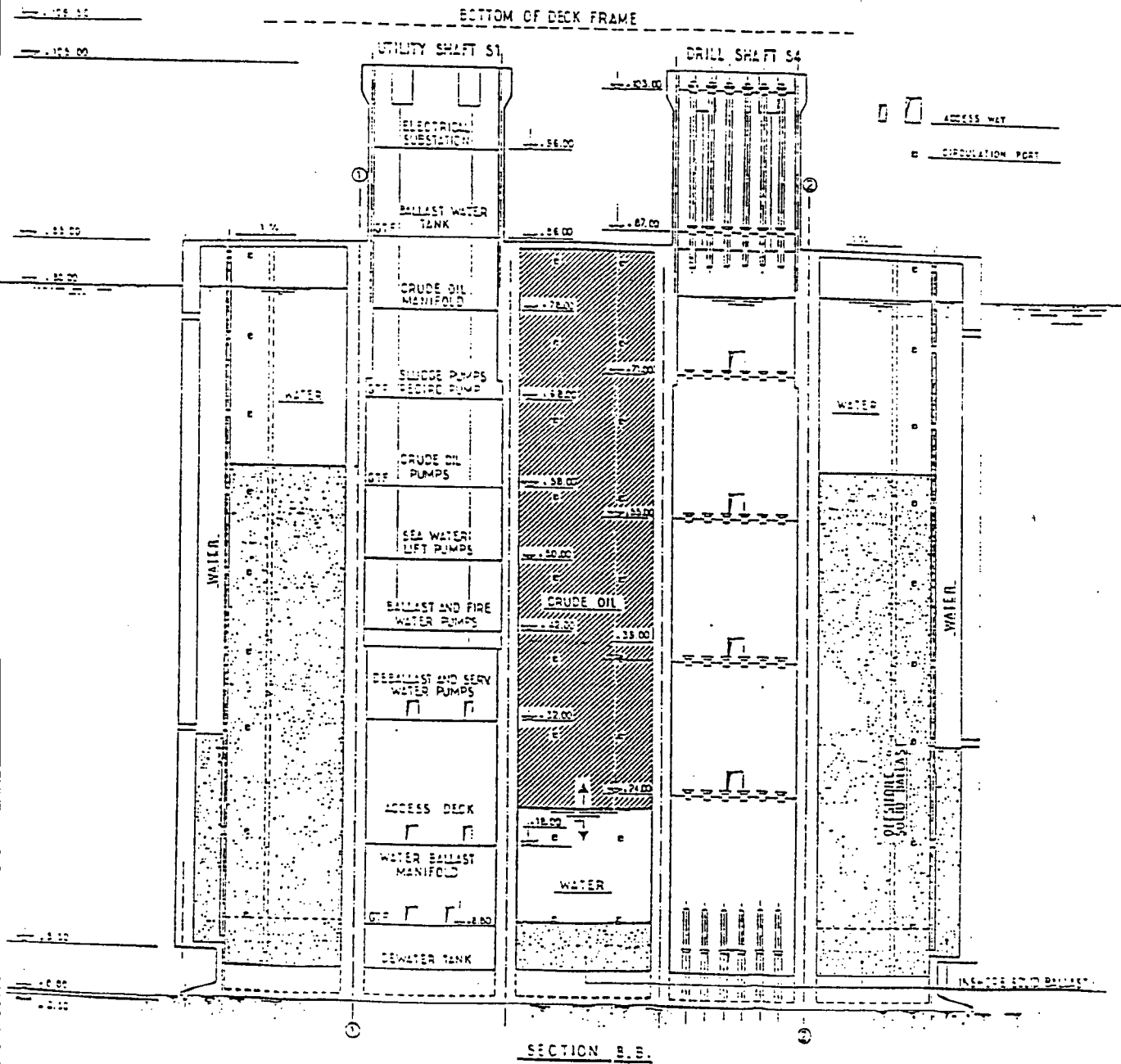
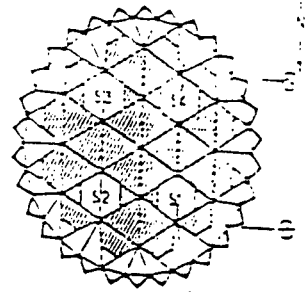


Figure 3.1 Elevation Section of Hibernia GBS 1986 Update Design

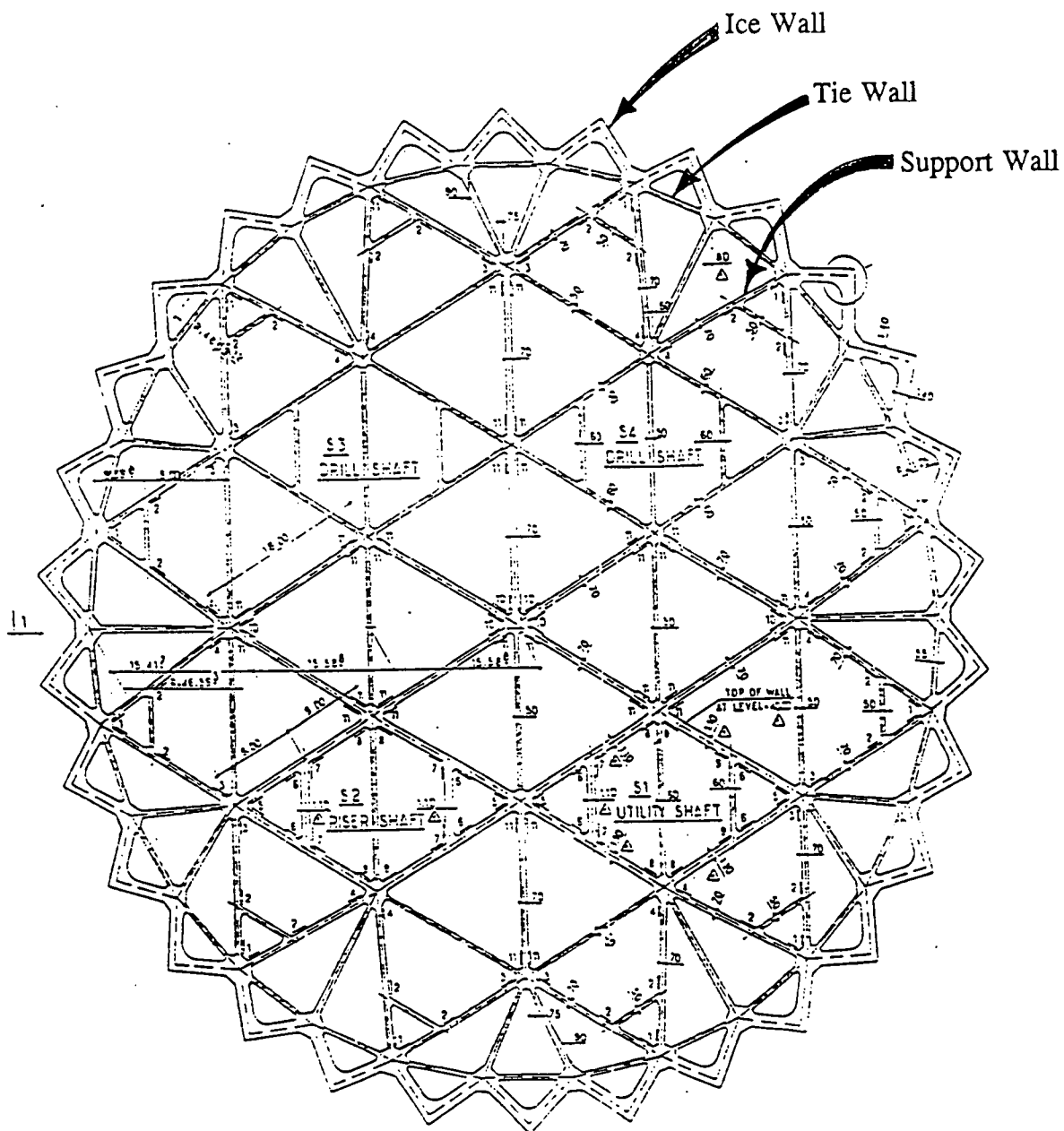


Figure 3.2 Plan View Cross Section of Hibernia GBS 1986 Update Design

4.0 DESIGN CRITERIA

4.1 CSA Safety Classes

Two safety classes are defined in Clause 4.5.2 of S471, which may be assigned to the structure as a whole or to its individual members or elements. The relevant table in S471, Table A2, is reproduced below to show the load factors and annual exceedance probability of loadings for the two safety classes.

| Description | Safety Class 1 | | Safety Class 2 | |
|--|--------------------------------------|-------------|--------------------------------------|-------------|
| | Annual exceedance probability, P_E | Load Factor | Annual exceedance probability, P_E | Load Factor |
| Specified loads, E_r , based on frequent environmental processes | 10^{-2} | 1.35 | 10^{-2} | 0.9 |
| Specified loads, E_r , based on rare environmental events | 10^{-4} to 10^{-3} | 1.0 | 10^{-2} | 1.0 |
| Specified accidental load, A | 10^{-4} to 10^{-3} | 1.0 | N/A | N/A |

Table 4.1 Annual Exceedance Probabilities and Load Factors for Specified Loads for CSA Safety Classes (from S471)

Further discussion on the classification of an individual element can be found in Clauses 8.4, 8.4.1, 8.4.2, and 8.4.3 of S474.1.

From the above information, the structure and ice wall have been assigned the following safety classes:

Structure: Safety Class 1

Ice Wall: Safety Class 1

Note that the Safety Class 1 structure as a whole has to be able to resist the 1 in 10,000 year iceberg, and as this has to be transferred through the ice wall and its immediate supporting structure anyway (in a damaged condition), there are no apparent savings in designating the ice wall as a Safety Class 2 element. The ice wall and its support walls must have sufficient strength or possess adequate energy-absorbing capacity in the ductile range to stop a 1 in 10,000 year iceberg before either oil storage is ruptured or any of the four towers supporting the topsides are intersected and human lives are put at risk. The shortest distance between the ice wall and a wall containing oil is 3.7 m, and so this distance is set as the maximum penetration distance of a 1 in 10,000 year iceberg after collapse of the ice wall.

4.2 CSA-S471 Load Combinations

For the principal operating phase loadings of waves and icebergs, the load combinations given in Table 6.2 of S471, for Safety Class 1, as they apply to the subject design problem, are given below:

4.2.1 Waves, a Frequent Environmental Process

| | | |
|---------------------------------------|---|---|
| ULS Load Combination 1 ⁽¹⁾ | = | 1.25 G _D ⁽²⁾ + 1.25 Q _B + 0.7 E _f |
| ULS Load Combination 2 | = | (1.05 or 0.9) G _D + 1.0 Q _B + 1.35 E _f |
| SLS/Fatigue Load Combination 9 | = | 1.0 G _D + 1.0 Q _B ± 0.77 E _f ⁽³⁾ |
| SLS/Local Damage Load Combination 9 | = | 1.0 G _D + 1.0 Q _B + 1.0 E _f |

where G_D = dead load from self weight of the concrete base

Q_B = solid ballast pressure only, as the outer ring of cells, which constitute the solid/water ballast cells, communicate with the sea and so the ice wall has no net hydrostatic pressure acting on it during the operation phase. Hydrostatic pressures during mating and installation have not been included in this design comparison as the pressures on the elements being used in this study were not supplied by Mobil, and are not expected to govern for the ice wall and immediately supporting walls in any event. The hydrostatic pressures during mating and installation will have the largest impact on the base slab immediately below the utility and riser shafts, which are dry at mating stage, but this part of the structure is not being designed in this study.

E_f = specified load from a 1 in 100 year maximum wave height of 1,769 MN force with a corresponding overturning moment of 79,544 MN-m. The amplitude of hydrodynamic pressure from the 1 in 100 year maximum wave is 241 kPa at the still water level and 143 kPa at the seabed.

4.2.2 Icebergs, a Rare Environmental Event

| | | |
|---|---|---|
| Ductility ULS Load Combination 3 | = | (1.05 or 0.9) G _D + 1.0 Q _B + 1.0 E _{r, 10000} |
| Local Damage Load Combination 10 ⁽⁴⁾ | = | 1.0 G _D + 1.0 Q _B + 1.0 E _{r, 100} |

where E_{r, 10000} = 1 in 10,000 year specified concentric iceberg load of 1,510 MN force with a corresponding overturning moment of 90,919 MN-m and a kinetic energy of 1,800 MJ, or an eccentric iceberg load with only one side of the gear teeth impacted by the iceberg.

E_{r, 100} = 1 in 100 year specified concentric iceberg load of 555 MN force with a corresponding overturning moment of 30,525 MN-m and a kinetic energy of 500 MJ, or an eccentric iceberg load with one side of the gear tooth loaded.

Notes:

1. Load combination numbers, except 10, are from Table 6.2 of S471.
2. It is assumed that the weight of the topsides is transferred down the towers/shafts and internal walls to the base slab of the GBS, for the purposes of this study, and do not effect the ice wall and its immediate supporting walls. The detailed design should incorporate a finite element analysis of the effect of varying topsides loads on the ice wall.
3. The maximum and minimum section forces used for the range of loads expected once every year as given in Clause 8.5.3 of S474, but can be taken as 0.7 times the 1 in 100 year loads as given in Clause 8.5.3 of S474.1. The maximum annual specified wave load is calculated as 1,335 MN with a corresponding overturning moment of 59,900 MN-m. The amplitude of hydrodynamic pressure from the annual maximum wave is 196 kPa at the still water level and 100 kPa at the seabed, or about 77% of the pressures from the 1 in 100 year maximum wave, and so the annual wave force is taken as 0.77 times the 1 in 100 year wave force.
4. A new load combination on local damage has been added to be consistent with Clause 9.6.1 of S474 and Clauses 8.4 and 9.6 of S474.1. We recommend that Table 6.2 of S471 be changed to include a new load combination for the 1 in 100 year iceberg.

The above quoted specified loads include companion processes as required in Table 6.1 of S471 and as developed in Section 4.0 of the G-2A Final Report. The maximum wave pressures are used in the design of the ice wall.

4.3 NPD Load Combinations

Sections 4.2.2 to 4.2.5 of NPD (1984) give the load coefficients for serviceability, fatigue, ultimate and progressive collapse limit states load combinations as:

| | | |
|---------------------------------------|---|-------------------------------|
| SLS Load Combination | = | 1.0 P + 1.0 L + 1.0 E + 1.0 D |
| Fatigue Limit States Load Combination | = | 1.0 P + 1.0 L + 1.0 E + 1.0 D |
| ULS Load Combination a | = | 1.3 P + 1.3 L + 0.7 E + 1.0 D |
| ULS Load Combination b | = | 1.0 P + 1.0 L + 1.3 E + 1.0 D |
| Progressive Collapse Load Combination | = | 1.0 P + 1.0 L + 1.0 E + 1.0 D |

- where
- P = permanent (solid ballast and self weight in this case) loads
 - L = variable functional loads (not included in this design since the ice wall is assumed to be isolated from the topsides and the oil storage cell pressures do not effect the ice wall design)
 - E = environmental loads
 - D = deformation loads (e.g. prestressing, shrinkage, temperature)

The following table (Table 4.2 of NPD, 1984) gives the following characteristic loads and load combinations for normal operations:

| Normal Operations | | | | | | |
|---------------------------|---------------------------------------|-----------------------|--|--|--|--|
| Loads | Serviceability Limit State | Fatigue Limit State | Ultimate Limit State | Limit State of Progressive Collapse | | |
| | | | | Abnormal Effect | Damaged Condition | |
| Permanent Loads | EXPECTED VALUE | | | | | |
| Variable Functional Loads | SPECIFIED VALUE | | | | | |
| Environmental Loads | Dependent on Operational Requirements | Expected Load History | Annual Probability of Exceedance = 10^{-2} | Annual Probability of Exceedance = 10^{-4} | Annual Probability of Exceedance = 10^{-2} | |
| Deformation Loads | EXPECTED VALUE | | | | | |
| Accidental Loads | Not Applicable | | | Annual Probability of Exceedance = 10^{-4} | Not Applicable | |

Table 4.2 NPD Loadings

4.4 Section Strength Design Criteria

4.4.1 CSA-S474

The material resistance factors are 0.67 for concrete, 0.85 for reinforcing bars, and 0.9 for prestressing tendons, as specified in Clause 7.4.1 of S474. As the 1 in 10,000 year iceberg is to be used for checking system ductility, and the 1 in 100 year iceberg will be used for checking local damage, waves shall be used as the primary loading for section strength design. The design loadings to be applied are given in the ULS Load Combinations 1 and 2 in Section 4.2.1 above.

4.4.2 NPD and NS 3473

Material resistance factors are 0.8 (1/1.25) for concrete and 0.87 (1/1.15) for the reinforcement, where the material coefficients given in Clause 10.4.3 of NS 3473, 1.25 for concrete and 1.15 for reinforcement, apply if the deviations in cross sectional dimensions are accounted for in the design, and as specified in Clause 34 of the 1991 unofficial NPD regulations. Both the 1 in 100 year wave and iceberg loads shall be used for designing the ice wall strength, using the ULS Load Combinations a and b given in Section 4.3 above.

4.5 Fatigue Design Criteria

4.5.1 CSA - S474

As given in Clause 7.4.1 of S474, material resistance factors of 1.0 shall be used in evaluating fatigue.

As given in Section 8.5 of S474, the fatigue life is to be checked using the maximum annual wave stress fluctuations to determine the range of stresses. If tensile stresses in the concrete exceed $0.25 f_{cr}$, the concrete shall be assumed to have cracked, and the stresses shall be calculated while neglecting concrete in tension.

As given in Clause 8.5.5 of S474, it may be assumed that a fatigue failure will not occur if the calculated stress ranges in the materials are less than the following values:

| Material | Value |
|--|-------------|
| Reinforcing Bars | |
| Straight | 140 MPa |
| Near bends or welds | 70 MPa |
| Prestressing tendons | |
| Wires or strands in straight ducts | 100 MPa |
| Wires or strands in curved ducts | 70 MPa |
| High strength bars | 60 MPa |
| Concrete in compression/compression | $0.40 f'_c$ |
| Concrete in compression/tension | $0.20 f'_c$ |

Table 4.3 Stress Range Limits for Checking Fatigue in S474

As given in Clause 8.5.6 of S474 and S474.1, when the stress ranges are greater than the values given in the above table, a cumulative fatigue damage calculation is required using Miner's rule:

$$\sum \frac{N_i}{N_{fi}} < 1.0$$

where N_i = number of cycles in the stress range i

N_{fi} = number of equal amplitude cycles in a stress range required to cause failure

The appropriate S-N curves, with the experimentally observed stress range divided by 1.5, are to be used.

4.5.2 NPD and NS 3473

As given in Clause 5.4 of NPD (1984), a material coefficient of 1.0 is to be used.

As given in Section 8.0 of Appendix 2 of NPD (1984), Miner's rule is to be used to estimate the fatigue life, with the number of load fluctuations multiplied by the factors given in the following table from Clause 5.4 of NPD (1984):

| Classification of structure components | Access for inspection and repair | | |
|---|----------------------------------|-----------------------|-----------------------|
| | No access or in the splash zone | Accessible | |
| | | Below the splash zone | Above the splash zone |
| Major importance for the structural integrity | 10 | 3 | 2 |

Table 4.4 Fatigue Factors in NPD

As given in A13.1.5 of NS 3473, the Miner's rule summation does not have to be made if the reinforcement stress range is less than 160 MPa for straight reinforcement.

4.5.3 Fatigue Loading

The amplitude of wave pressures at the still water level versus the number of cycles per year are given in the following table:

| Wave Category Number | Individual Wave Pressure Amp. at SWL (kPa) | Number of Cycles Per Year |
|----------------------|--|---------------------------|
| 1 | 5 | 2,778,000 |
| 2 | 15 | 1,641,000 |
| 3 | 25 | 917,000 |
| 4 | 35 | 490,000 |
| 5 | 45 | 245,000 |
| 6 | 55 | 115,000 |
| 7 | 65 | 52,000 |
| 8 | 75 | 23,000 |
| 9 | 85 | 9,800 |
| 10 | 95 | 4,300 |
| 11 | 105 | 1,900 |
| 12 | 115 | 860 |
| 13 | 125 | 400 |
| 14 | 135 | 190 |
| 15 | 145 | 90 |
| 16 | 155 | 40 |
| 17 | 165 | 20 |
| 18 | 175 | 9 |
| 19 | 185 | 4 |
| 20 | 195 | 2 |

Table 4.5 Pressure Amplitude versus Number of Cycles for Fatigue at Hibernia

This table was developed using the data presented in the G-2A report. The range of stresses in the ice wall are calculated as the product of the stress range from the annual maximum wave, multiplied by the ratio of the individual wave pressures to the annual maximum wave pressure of 196 kPa at the still water level.

4.6 Ductility Criteria

4.6.1 CSA-S474

As given in Clause 7.4.1 of S474, material resistance factors of 0.90 for concrete and 1.0 for the reinforcing and prestressing steel are to be applied for evaluating system ductility.

The load combination for which system ductility is to be checked is a 1 in 10,000 year iceberg using the ULS Load Combination 3 as given in Section 4.2.2 above.

As given in Clause 8.4.4 of S474, it may be assumed that an element possesses adequate energy-absorbing capacity, if it can be documented that the load deformation response of the element is such that at a deformation of six times the yield deflection the resistance still exceeds 50% of the maximum resistance.

As given in Clause 8.4 of S474.1, system ductility may be checked using a simplified failure model of the structure. The deflections during indentation by a 1 in 10,000 year iceberg shall be less than about 3.7 m to ensure that the oil storage compartments are not ruptured and the internal shafts supporting the towers and superstructure are not damaged.

4.6.2 NPD and NS 3473 (Limit State of Progressive Collapse)

As given in Clause 5.6 of NPD (1984), material resistance factors of 1.0 are to be applied for the progressive collapse limit state.

The load combination for the 1 in 10,000 year iceberg is given in Section 4.3 above, for which the load coefficients are all 1.0.

4.7 Serviceability Criteria

4.7.1 CSA - S474

As given in Clause 7.4.2 of S474, material resistance factors of 1.0 are to be applied for the serviceability limit state.

There are two criteria to be checked:

- Crack widths at the surface
- Local damage

Crack Widths

Using the SLS Load Combination number 9 given in Section 4.2.1 above, with the environmental load being from the hydrodynamic wave pressure from the maximum annual wave and 1.0 load factors (or a 0.77 load factor on the maximum 1 in 100 year pressure in this case), the average crack widths on the surface of the element, as specified in Clause 9.3.2 of S474, shall not exceed:

- a. in the splash zone, 0.15 mm
- b. in other zones, 0.3 mm

Local Damage

Using the Local Damage Load Combination number 10 given in Section 4.2.2 above, with the environmental load being the 1 in 100 year iceberg (the worst condition for this case), with load factors of 1.0, as specified in Clause 7.4.2 of S474, the tensile stresses in the reinforcing bars shall not exceed 0.9 f_y and compressive stresses in the concrete shall not exceed 0.67 f'_c , as specified in Clause 9.6.2. of S474.

Concrete Cover

The minimum concrete cover over the principal reinforcement, given in Table 5.3 of S474, is 65 mm in the splash zone and 50 mm otherwise, and not less than 1.5 times the effective diameter of the reinforcing bars as specified in Clause 5.4.1 of S474.

The minimum concrete cover to prestressing tendons is 90 mm in the splash zone and 75 mm otherwise, as given in Clause 5.4.1 of S474.

4.7.2 NPD and NS 3473

As given in Section 10.4.3 of NS 3473, material resistance factors of 1.0 are to be applied for the serviceability limit state.

The nominal characteristic crack widths at the level of the reinforcement, as specified in Clause 15.2.2. of NS 3473, and modified by Section 5.3 of the concrete appendix to NPD (1991), shall be limited to:

- a. 0.25 mm⁽¹⁾ for elements permanently submerged in saline water (Class NA) for reinforcement sensitive to corrosion (eg. prestressing steel) and 0.1 mm in the splash zone (Class MA).
- b. 0.5 mm⁽¹⁾ for elements permanently submerged in saline water (Class NA) for reinforcement slightly sensitive to corrosion (eg. ordinary reinforcing steel), and 0.2 mm in the splash zone (Class MA).

Note: These crack widths are to be reduced by 0.1 mm if the stresses in the reinforcing steel exceeds 0.7 fy.

As given in Clause 15.2.4 of NS 3473, "the magnitude of the short-term loads shall be chosen such that the crack width criterion will not be exceeded more than 100 times during the design life of the structure", where the design life is 18 years. The magnitude of the wave force that is exceeded 100 times in 18 years is calculated as 74% of the 1 in 100 year wave force.

The minimum concrete cover requirements for ordinary reinforcing steel, as given in Clause 17.1.8 of NS 3473, are:

- a. 50 mm in the splash zone
- b. 40 mm below the splash zone

4.8 Minimum Reinforcement Requirements

4.8.1 CSA-S474

As given in Clause 10.2.1 of S474, the area of reinforcement near each face and in each of the two orthogonal reinforcement directions shall not be less than .003 times the area of the concrete section, nor shall it be less than the area calculated using the following equation:

$$A_s = (f_{cr} + w) bh_{ef}/f_y$$

- where:
- A_s = the area of reinforcement within the effective embedment thickness
 - f_{cr} = cracking strength of concrete = $0.4 \sqrt{f'_c}$
 - w = the fluid pressure on the face
 - b = the width of the section
 - h_{ef} = effective embedment thickness (see Clause 9.3.5 of S474)
 - f_y = reinforcement yield strengths

4.8.2 NPD and NS 3473

As given in Section 7 of the concrete appendix to NPD (1991), the minimum reinforcement in walls and shells, on each side and in each direction is:

$$A_s = kA_c (f_{tk} + w)/f_{yk}$$

- where:
- A_s = area of reinforcement
 - k = 0.40 for $h \leq 0.3$ m
 - k = $0.49 - 0.3 h$ for 0.3 m $< h \leq 0.8$ m
 - k = 0.25 for $h > 0.8$ m
 - A_c = area of concrete, not reduced by the area of steel
 - f_{tk} = expected lower characteristic tensile strength of the concrete
= 3.51 MPa for concrete with f'_c of 50 MPa (or f_{ck} using NS 3473 terminology).
 - f_{yk} = reinforcement yield strength = 400 MPa
 - w = the fluid pressure on the face

4.9 Summary of Design Criteria

The design criteria for the CSA and NPD codes are summarized in the following tables.

| Design Loading | Material Resistance Factors | Load Combinations |
|--|--|--|
| ULS Strength Design $E_f = 1$ in 100 year wave | $\phi_c = 0.67$ $\phi_r = 0.85$ $\phi_p = 0.9$ | $1.25 G_D + 1.25 Q_B + 0.7 E_f$ $(1.05 \text{ or } 0.9) G_D + 1.0 Q_B + 1.35 E_f$ |
| ULS Ductility $E_f = 1$ in 10,000 year iceberg | $\phi_c = 0.9$ $\phi_s = 1.0$ | $(1.05 \text{ or } 0.9) G_D + 1.0 Q_B + 1.0 E_f$ |
| ULS Fatigue $E_f =$ annual wave (77% of pressures of 1 in 100 year wave) or summation using Miner's Rule and a factor of safety of 1.5 on stresses | $\phi_c = 1.0$ $\phi_s = 1.0$ | $1.0 G_D + 1.0 Q_B + 1.0 E_f$ |
| SLS Crack Width $E_f =$ annual wave Crack widths are 0.15 mm in the splash zone and 0.3 mm otherwise | $\phi_c = 1.0$ $\phi_s = 1.0$ | $1.0 G_D + 1.0 Q_B + 1.0 E_f$ |
| SLS Local Damage $E = 1$ in 100 year wave or iceberg | $\phi_c = 1.0$ $\phi_s = 1.0$ | $1.0 G_D + 1.0 Q_B + 1.0 E$ |

Table 4.6 CSA Design Criteria Summary

| Design Loading | Material Resistance Factors | Load Combinations |
|--|-----------------------------------|--|
| ULS Strength Design $E_f = 1$ in 100 year wave $E_r = 1$ in 100 year iceberg $E = E_f$ or E_r | $\phi_c = 0.8$ $\phi_r = 0.87$ | $1.3P + 0.7E + 1.0D$ $1.0P + 1.3E + 1.0D$ |
| Limit State of Progressive Collapse $E_r = 1$ in 10,000 year iceberg | $\phi_c = 0.9$ $\phi_s = 1.0$ | $1.0P + 1.0E_r + 1.0D$ |
| Fatigue Limit State Miner's Rule with 10 times number of cycles in splash zone or inaccessible and 3 times number of cycles below splash zone. | $\phi_c = 1.0$ $\phi_r = 1.0$ | $1.0P + 1.0 E_f + 1.0D$ |
| SLS Crack Width $E_f = 100$ waves in 18 years with a crack width of 0.2 mm in the splash zone and 0.5 mm below the splash zone. | $\phi_s = 1.0$ $\phi_s = 1.0$ | $1.0P + 1.0 E_f + 1.0D$ |

Table 4.7 NPD Design Criteria Summary

The material resistance factor symbols used in the above tables are:

- ϕ_c = concrete material resistance factor
- ϕ_r = reinforcement material resistance factor
- ϕ_p = prestressing steel resistance factor
- ϕ_s = general steel resistance factor

The environmental force/event symbols used in the above tables are:

- E_f = specified load based on a frequent environmental process
- E_r = specified load based on a rare environmental event

The remaining loading symbols used in the above tables are defined in Sections 4.2 and 4.3 above, for the CSA and NPD codes, respectively.

5.0 ANALYSIS

5.1 Finite Element Models

Two finite element models were constructed using COSMOS/M Version 1.61, namely fine and coarse mesh models.

5.1.1 Fine Mesh Model

The fine mesh model was constructed with sufficient detail so that the ice wall, tie wall and the supporting walls immediately behind them could be designed to a preliminary design level. A portion of the structure was removed from the whole structure; five elements on each side of each gear tooth are modelled and the structure is supported by springs on the back wall as shown in Figure 5.1. The fine mesh model is shown in perspective in Figure 5.2.

The model was divided into various levels so that the iceberg loading and resulting stresses in the structure would be accurately simulated. The various levels, numbers of elements in each level, and element sizes are given in Table 5.1.

| Level | Distance from Base | Number of Elements Vertically | Element Size |
|-------|--------------------|-------------------------------|--------------|
| 1 | 0 to 46 m | 6 | 7.67 m |
| 2 | 46 to 72 m | 6 | 4.33 m |
| 3 | 72 to 85 m | 2 | 6.50 m |

Table 5.1 Levels of Element Sizes in Fine Mesh Model

Each "gear tooth" of the ice wall is divided into 10 elements and the respective aspect ratios of the elements at the various levels are:

| <u>Level 1</u> | <u>Aspect Ratio</u> |
|----------------|---------------------|
| 1 | 5.56 |
| 2 | 3.14 |
| 3 | 4.72 |

For accurate results, the element aspect ratio should not be more than about 5. Since only elements at Level 1 have an aspect ratio slightly above 5, while the more heavily loaded elements in Level 2 and 3 are within the aspect ratio of 5, the results are considered acceptable.

The fine mesh finite element model has the following attributes:

- 2,441 nodes
- 3,068 elements
- 11,970 degrees of freedom

5.1.2 Coarse Mesh Model

The coarse mesh model simulates the whole GBS structure and was modelled to reflect accurately the effect of wave loading on the elements around the entire structure, in particular the ice wall near the base slab. The principal reason for making this model is to evaluate fatigue effects from wave loading.

The structure is divided up into 8 elements vertically in the ice wall, as shown in Figure 5.3.

The coarse mesh finite element model has the following attributes:

- 1,140 nodes
- 2,109 elements
- 5,130 degrees of freedom

5.2 Load Cases and Combinations

The load cases and combinations that are applied to the structure for the design of the ice wall and the immediately supporting walls, to the CSA and NPD codes for the operating phase of the structure, are given in Table 5.2.

| Load Combination Numbers in Computer Analysis | Description | Load Cases | | | | | | | | | | | | |
|---|---|------------|---------------|----------------------|-------------|-------------------------------------|------------------------------------|-----------------------------------|------------------------------------|-----------------------------------|------|--|------|------|
| | | Gravity | Solid Ballast | 10 ² Wave | Annual Wave | Wave Exceeded 100 times in 18 years | 10 ³ Concentric Iceberg | 10 ³ Eccentric Iceberg | 10 ⁴ Concentric Iceberg | 10 ⁴ Eccentric Iceberg | | | | |
| <u>CSA</u> | | | | | | | | | | | | | | |
| 21 | ULS Load Combination 1 | 1.25 | 1.25 | 0.70 | | | | | | | | | | |
| 22 | ULS Load Combination 2 | 1.05 | 1.00 | 1.35 | | | | | | | | | | |
| 24 | SLS/Fatigue Load Combination 9 ⁽¹⁾ | 1.00 | 1.00 | | ± 1.00 | | | | | | | | | |
| 25 | ULS Load Combination 3 | 1.05 | 1.00 | | | | | | | | | | 1.00 | |
| 26 | Local Damage Load Combination 10a | 1.00 | 1.00 | | | | | 1.00 | | | | | | |
| 32 | ULS Load Combination 3 | 1.05 | 1.00 | | | | | | | | | | | 1.00 |
| 34 | Local Damage Load Combination 10b | 1.00 | 1.00 | | | | | | | | 1.00 | | | |
| <u>NPD</u> | | | | | | | | | | | | | | |
| 27 | SLS Load Combination | 1.00 | 1.00 | | | 1.00 | | | | | | | | |
| 28 | ULS Load Combination A | 1.30 | 1.30 | 0.70 | | | | | | | | | | |
| 29 | ULS Load Combination B | 1.00 | 1.00 | 1.30 | | | | | | | | | | |
| 30 | Progressive Collapse | 1.00 | 1.00 | | | | | | | | | | 1.00 | |
| 33 | Progressive Collapse | 1.00 | 1.00 | | | | | | | | | | | 1.00 |
| 35 | ULS Load Combination B for Iceberg | 1.00 | 1.00 | | | | | 1.30 | | | | | | |
| 36 | ULS Load Combination B for Iceberg | 1.00 | 1.00 | | | | | | | | 1.30 | | | |

Table 5.2 Load Combinations for Design of Hibernia GBS Ice Wall and Supporting Walls Using CSA and NPD Codes

Notes: 1. This load combination is used for both CSA and NPD designs.

5.3 Analysis Results

The Von Mises stress plots for the ice wall for load combinations 23 (annual wave), 25 (concentric 1 in 10,000 year iceberg), and 32 (eccentric 1 in 10,000 year iceberg), are shown in Figures 5.4, 5.5. and 5.6, respectively. It can be seen that the maximum stresses from the eccentric iceberg impact are about the same magnitude as the maximum stresses from the concentric iceberg impact, even though the load of the eccentric impact is only 50% of the concentric impact load. Due to high tensions induced in the ice wall from the eccentric impact, it is this load combination which generally governs in the design of the ice wall, as noted in the next section.

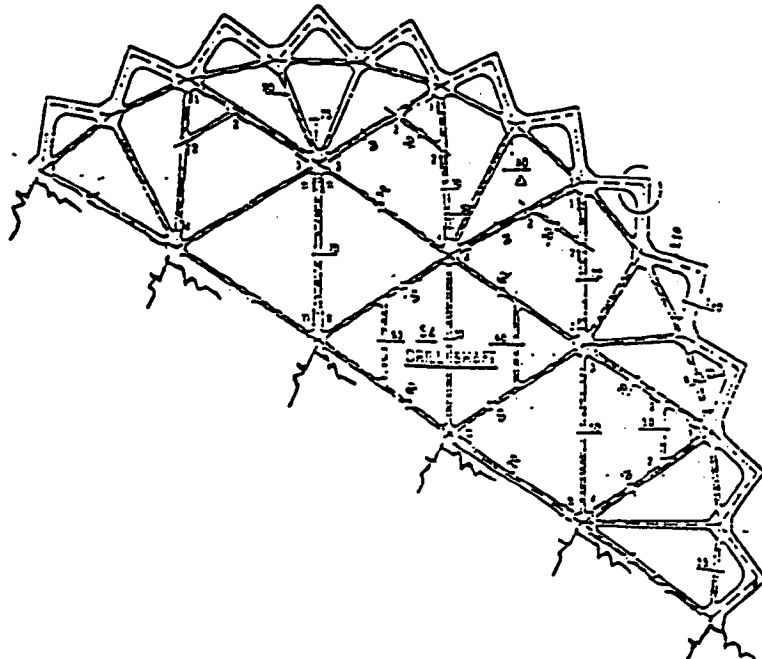


Figure 5.1 Portion of Structure Removed for Fine Mesh Finite Element Model

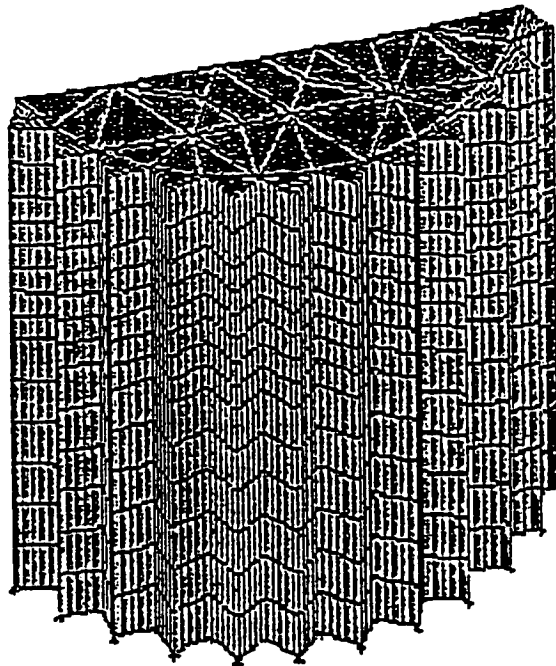


Figure 5.2 Fine Mesh Finite Element Model

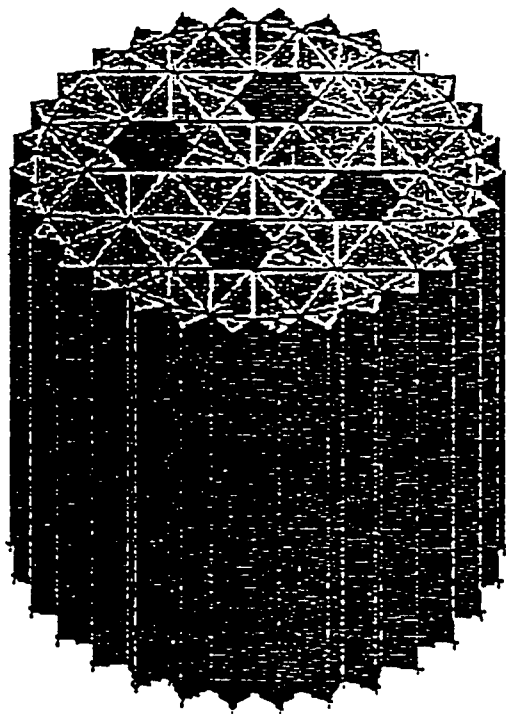


Figure 5.3 Coarse Mesh Finite Element Model

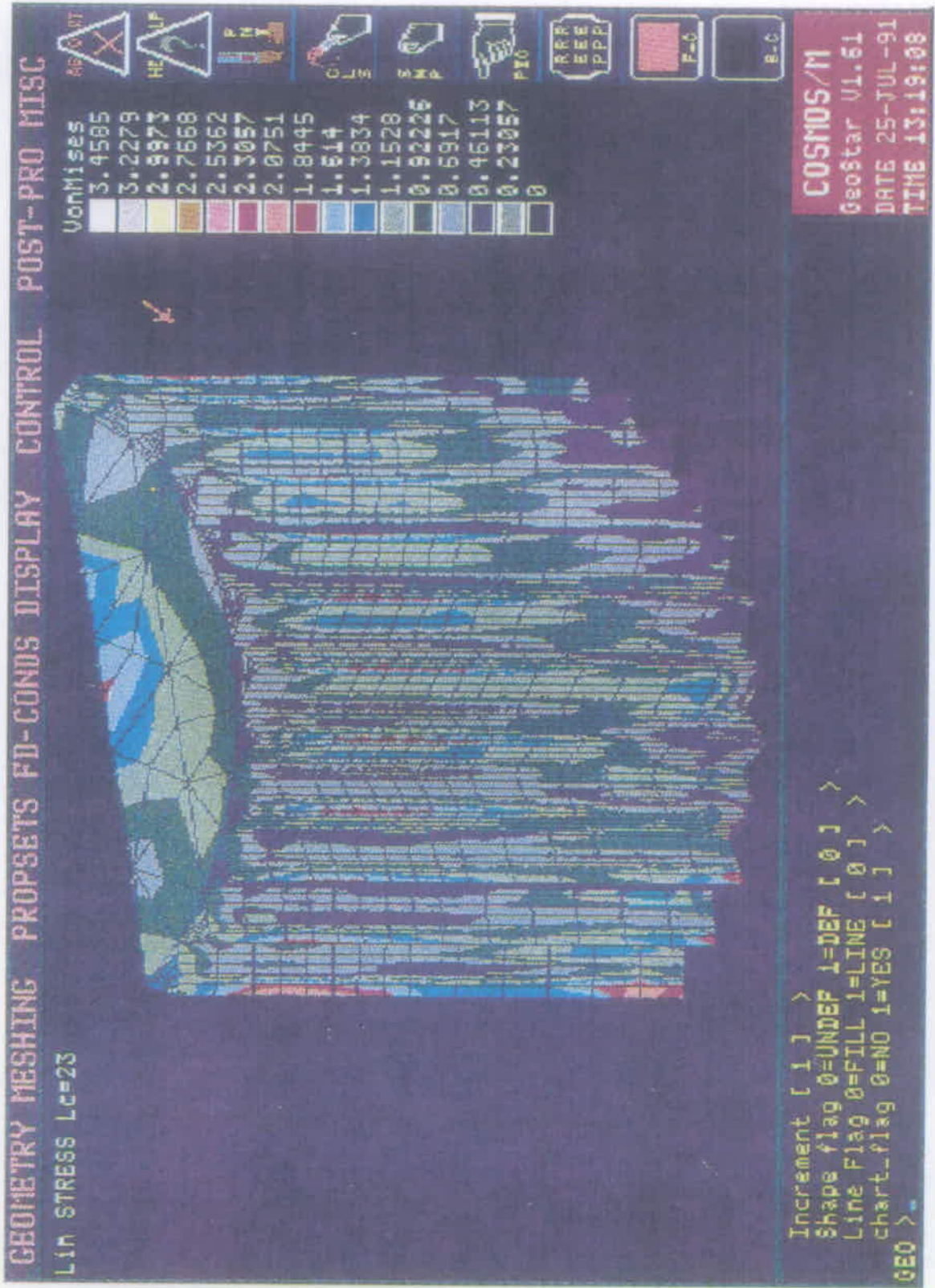


Figure 5.4 Von Mises Stress Plot for Annual Wave Load Combination 23

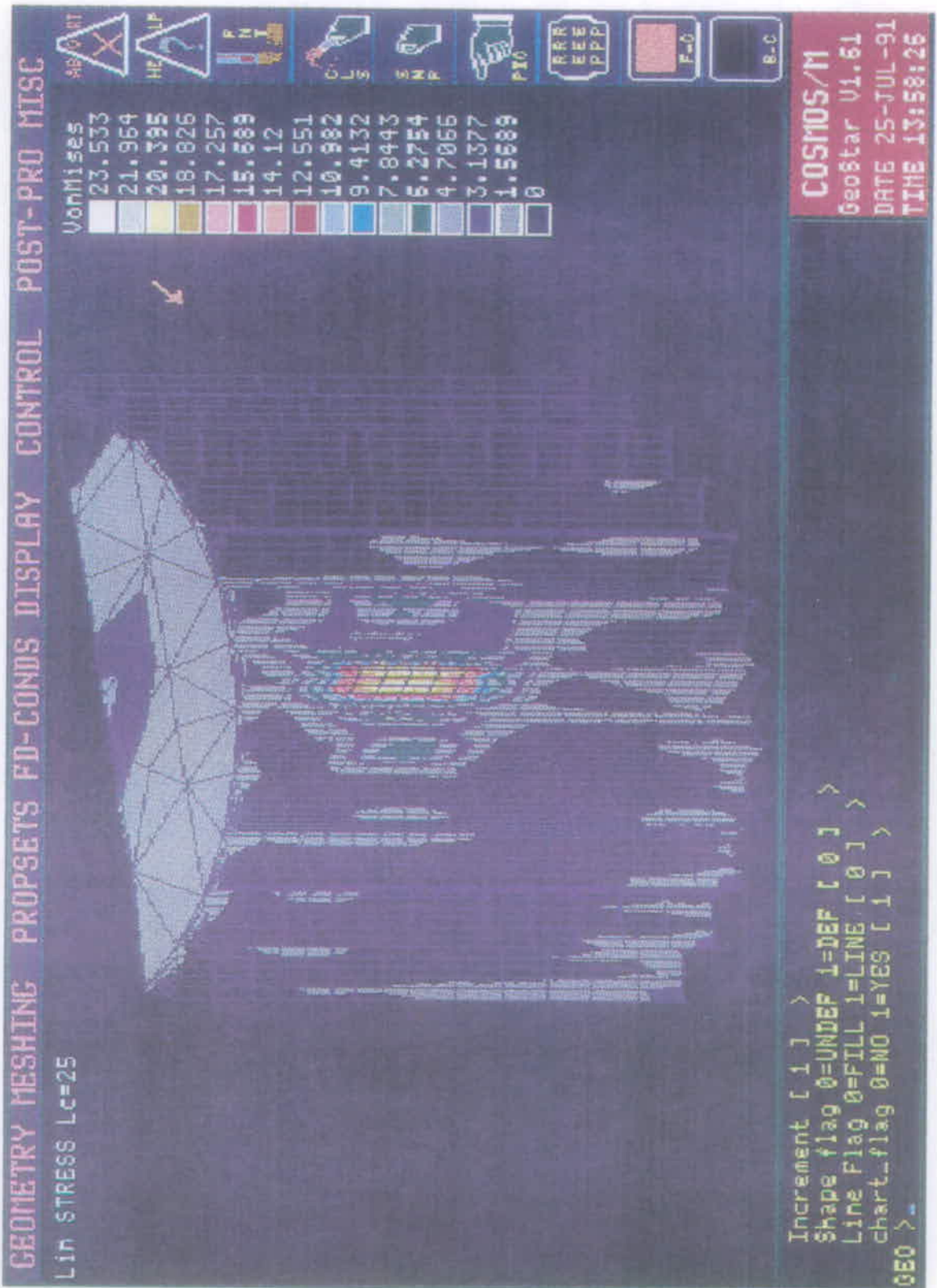


Figure 5.5 Von Mises Stress Plot for 1 in 10,000 year Concentric Iceberg Impact

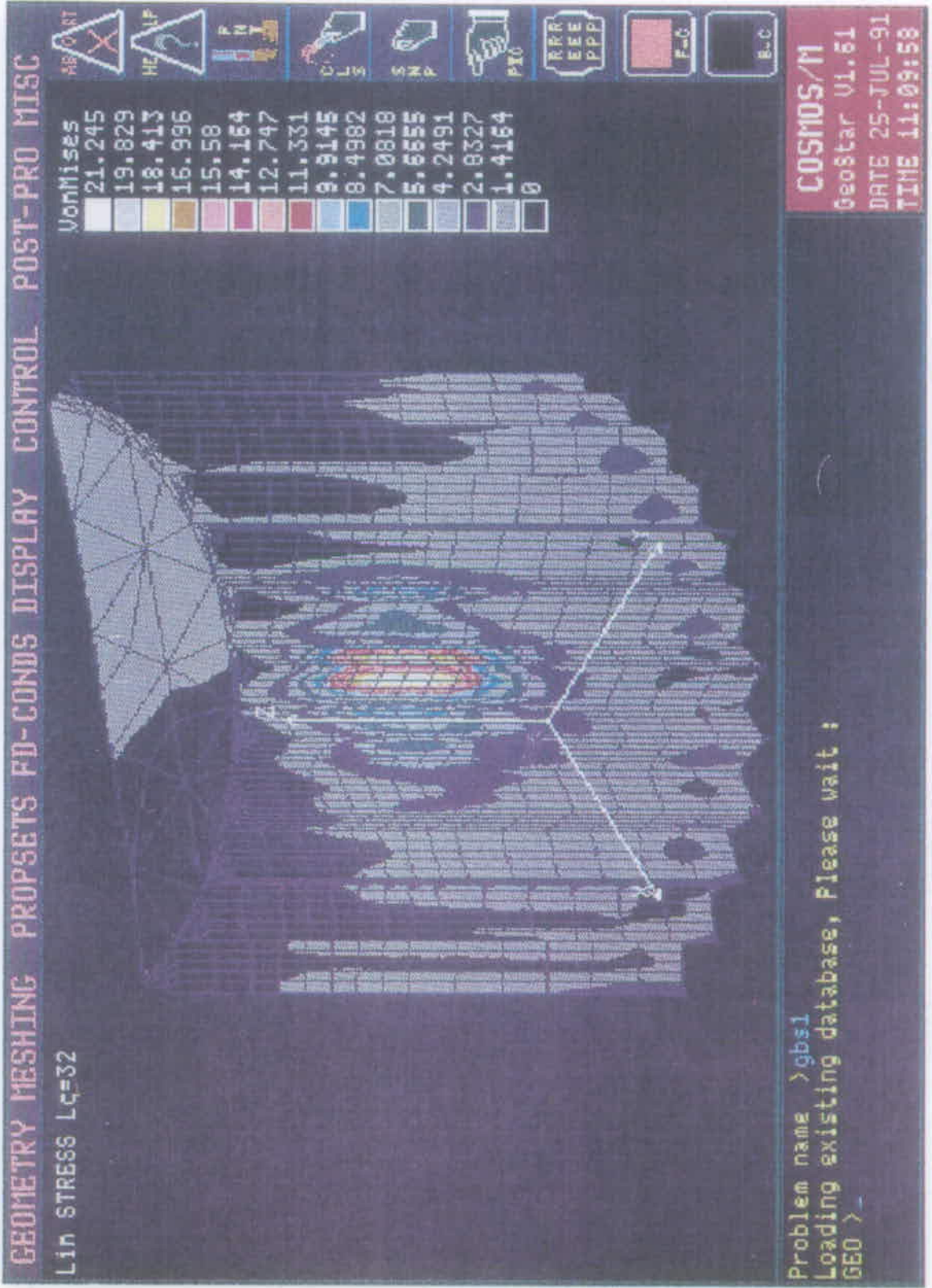


Figure 5.6 Von Mises Stress Plot for 1 in 10,000 Year Eccentric Iceberg Impact

6.0 DESIGN

The designs of the ice, tie, and support walls were initially performed by determining the reinforcing steel needed to meet the strength and serviceability requirements of the two design codes. During the initial design process, the detailing requirements of concrete cover and distance between reinforcing bars and bundles were accounted for. The final reinforcement layout includes the requirements of minimum reinforcement and constructability.

The designs of the critical elements are presented in this report as follows:

1. Design calculations were made for the reinforcement required to meet strength and serviceability requirements.
2. The bar size and spacing changes required to meet the minimum reinforcement and constructability requirements are given in Tables 6.1 to 6.6.
3. The final design configurations including minimum reinforcement are shown in Figures 6.1 to 6.6.

6.1 Ice Wall

6.1.1 Design of Ice Wall Using CSA-S474

The design calculations are summarized in a local damage run for an eccentric impact from a 1 in 100 year iceberg using the SLS option in SHELL474 for element 1108. For this run the cracking strength of the concrete is reduced to a low level so that the stress in the reinforcement that is printed out in the output is at the location of a crack. The maximum horizontal (x) reinforcement stress is 357.8 MPa, which is just under the 360 MPa allowed of (0.9 fy).

A second run was made on the local damage load combination for element 1108, but with the following changes:

- Using the ULS option in SHELL474, with ULS material factors.
- The input file was changed, as given in Table 6.1, to be consistent with the final design shown on Figure 6.1.

| Item | Changes | | Reason |
|---------------------------------|------------------|----------------------|------------------|
| | From | To | |
| Horizontal Reinforcing Exterior | 35 M @ 300 | 30 M @ 125 | .003 Area Rule |
| Vertical Reinforcing Both Sides | 25 M @ 300 | 35 M @ 235 | .003 Area Rule |
| Prestressing | 13 strands @ 600 | 10 strands @ 470 | Constructability |
| Transverse Reinforcing | 10 M @ 150 Grid | 15 M @ 235 H x 125 V | Constructability |

Table 6.1 Changes in Reinforcement and Prestressing in the Ice Wall Designed to S474 to Achieve Minimum Reinforcement and Constructability Requirements, from the Design for Strength and Serviceability Requirements Only (dimensions are in mm).

The results from this run show a load factor of only 1.1, which appears low for a 1 in 100 year iceberg load case where the load is unfactored; this can be compared with a 1 in 100 year wave where the critical calculated load factor is 1.88 for the ULS load combination where the wave force is factored by 1.35.

It would appear that the selection of a 1 in 100 year iceberg for performing a local damage design calculation may not be adequate, and the suggestion is put forward that a longer return period iceberg be used for this purpose as determined by a risk/reliability study.

6.1.2 Design of Ice Wall Using NPD/NS 3473

The transverse shear steel is designed using the spreadsheet NPDIW, which incorporates the provisions in Section 12.3.2 of NS 3473. Transverse shear reinforcement of $5,000 \text{ mm}^2/\text{m}^2$ is calculated using this method. This corresponds to the critical case of the ULS load combination of a 1 in 100 year eccentric iceberg impact for element 1109.

Because the principles used in SHELL474 meet the requirements of Section 12.5.1 of NS 3473, in-plane reinforcement is designed using SHELL474, with the following adjustments made to the input:

- The shear forces are set to zero, as the transverse steel is already designed using the simplified method.

- Additional tensile forces are added to the input load vectors to account for the effect of the out-of-plane shear forces on the longitudinal steel, as given in Section 12.3.4 of NS 3473.

The critical load factor is 1.016 for element 1109.

The changes to the reinforcement to meet minimum reinforcement and constructability requirements are given in Table 6.2. The final design of the ice wall to NPD/NS 3473 is shown in Figure 6.2.

| Item | Changes | | Reason |
|---------------------------------|---------------------|-------------------------|------------------|
| | From | To | |
| Horizontal Reinforcing Exterior | 35 M @ 300 | 30 M @ 145 | Constructability |
| Vertical Reinforcing Both Sides | 25 M @ 300 | 35 M @ 250 | Min. Reinf. Eqn. |
| Transverse Reinforcing | 15 M @ 200 Grid | 15 M @ 250 H x 145 V | Constructability |
| Prestressing | 13 Strands @ 600 | 11 Strands @ 500 | Constructability |

Table 6.2 Changes in Reinforcement and Prestressing in the Ice Wall Designed to NPD/NS 3473, to Achieve Minimum Reinforcement and Constructability Requirements, from the Design for Strength and Serviceability Requirements Only (dimensions are in mm).

6.2 Tie Wall

6.2.1 Design of Tie Wall Using CSA-S474

The design calculations were made for the following conditions:

- Crack width calculations for element 1819 for the annual wave SLS load combination using a spreadsheet called CSATW which uses input from SHELL474. The spreadsheet was required as the crack widths in the earlier versions of SHELL474 were found to be slightly different from those using the procedure set out in Section 9.3 of S474. The most recent version of SHELL474 has the same crack width calculations as S474 and the CSATW spreadsheet. The crack widths calculated were 0.281 mm and .285 mm (0.3 mm is allowed) at the outside and inside surfaces, respectively.
- A ULS run of SHELL474 on the 1 in 100 year wave with a load factor of 1.35 for element 1851. The maximum load factor calculated is 1.11, where 1.00 is required.

The local bending moments and shears from the solid ballast, calculated accounting for silo theory, were added to the finite element results to calculate both crack widths and the ULS.

The changes to the reinforcement to meet minimum reinforcement and constructability requirements are given in Table 6.3. The final design of the tie wall to S474 is shown in Figure 6.3.

| Item | Changes | | Reason |
|---------------------------------|-----------------|------------------------------|------------------|
| | From | To | |
| Horizontal Reinforcing Exterior | 35 M @ 190 | 35 M @ 150 | Constructability |
| Vertical Reinforcing | 25 M @ 300 | 30 M @ 275 | Min. Reinf. Eqn. |
| Transverse Reinforcing | 10 M @ 150 Grid | 10 M @ 137.5 H x 150 V | Constructability |
| Prestressing Steel | 5 Strands @ 600 | 5 Strands @ 550 | Constructability |

Table 6.3 Changes in Reinforcement and Prestressing in the Tie Wall Design to S474, to Achieve Minimum Reinforcement and Constructability Requirements, from the Design for Strength and Serviceability Requirements Only (dimensions are in mm).

6.2.2 Design of Tie Wall Using NPD/NS 3473

The design of the tie wall for the crack width criteria involved the following procedure:

1. Make a SLS run on SHELL474.
2. Extract the strain, stress and crack orientation required in Section 15.6 of NS 3473.
3. Run the ULS option of SHELL474 with 10% of the input load vector, and determine the cracking load, using the display option of SHELL474.
4. Run the SLS option of SHELL474 with a small value of tensile strength of concrete to determine the stress in the reinforcement at the crack for both the cracking load and the full SLS load.
5. Calculate the factor "r" in Clause 15.6.2 of NS 3473 using $k_1 = 0.4$ (for ribbed bars) and $\beta = 0.5$ (for repeated loads).
6. Calculate the characteristic crack width at the level of the reinforcement using the spreadsheet NPDCRT. Note that the average crack width at the surface using CSA-S474 is also printed out for comparison.

The design calculations included:

- A SLS run of SHELL474 for the load vectors on element 1819 for the wave that is exceeded 100 times in 18 years, modified for the shear and bending moment from the solid ballast pressure calculated accounting for silo theory, at the centre and inflection point.
- Crack width calculations which yield crack widths of .462 mm and .468 mm at the centre and inflection points, respectively. Note that the CSA crack width at the surface is about 40% smaller, as calculated in these spreadsheets.
- A ULS run of the element and load combination with the lowest load factor, namely element 1844 in the 100 year wave. The calculated minimum load factor for this case is 1.08. Note that the lowest calculated load factor for element 1851, which governs for the CSA design, is 1.11.

The changes to the reinforcement to meet minimum reinforcement and constructability requirements are given in Table 6.4. The final design of the tie wall to NPD/NS 3437 is shown in Figure 6.4

| Item | Changes | | Reason |
|------------------------------------|-----------------|-------------------------|------------------|
| | From | To | |
| Transverse Reinforcing | 10 M @ 150 Grid | 10 M @ 125 H x 165 V | Constructability |
| Vertical Reinforcing | 25 M @ 300 | 25 M @ 250 | Min. Reinf. Eqn. |
| Horizontal Reinforcing Interior | 35 M @ 200 | 35 M @ 165 | Constructability |
| Prestressing | 5 Strands @ 600 | 4 Strands @ 500 | Constructability |

Table 6.4 Changes in Reinforcement and Prestressing in the Tie Wall Design to NPD NS 3473, to achieve minimum reinforcement and constructability requirements, from the Design for Strength and Serviceability Requirements Only (dimensions are in mm).

6.3 Support Wall

6.3.1 Design of Support Wall Using CSA-S474

The design calculations were made for the following conditions:

- Crack width calculations for element 2010 for the annual wave SLS load combination, including a run of SHELL474 and the crack width calculation spreadsheet CSACRS, which calculated average crack widths at the surface using the procedure given in Section 9.3 of S474. The calculated crack width is 0.296 mm, where 0.3 mm is allowed.
- A ULS run of SHELL474 on the 100 year wave with a load factor of 1.35 for element 2016. The maximum load factor calculated is 1.20, where 1.00 is required.

The changes to the reinforcement and prestressing to meet minimum reinforcement and constructability requirements are given in Table 6.5. The total design of the support wall to S474 is shown in Figure 6.5.

| Item | Changes | | Reason |
|------------------------|--------------------|----------------------------|------------------|
| | From | To | |
| Vertical Reinforcing | 25 M @ 300 | 35 M @ 300 | Min. Reinf. Eqn. |
| Transverse Reinforcing | 10 M @ 150 Grid | 10 M @ 150 H x 135 V | Constructability |

Table 6.5 Changes in the Reinforcement and Prestressing in the Support Wall Design to S474, to Achieve Minimum Reinforcement and Constructability Requirements, from the Design for Strength and Serviceability Requirements Only (dimensions are in mm).

6.3.2 Design of Support Wall Using NPD/NS 3473

The design calculations were made for the following conditions:

- Crack width calculations for element 2010 for the wave that is exceeded 100 times in 18 years. The procedure for calculating crack widths is outlined in Section 6.2.2 above. the SLS run on SHELL474 is presented, along with the crack width spreadsheet NPDCRS. The calculated crack width is 0.498 mm, where 0.5 mm is allowed.
- A ULS of the element and load combination with the lowest load factor, namely element 2016 in the 100 year wave load combination with a 0.7 load factor on the wave load and 1.3 load factors on the structure self weight and solid ballast. The load factor for this case is 1.125.
- The changes is the reinforcement and prestressing to meet minimum reinforcement and constructability requirements are given in Table 6.6. The final design of the support wall to NPD/NS 3473 is shown in Figure 6.6

| Item | Changes | | Reason |
|------------------------|-----------------|-------------------------|------------------|
| | From | To | |
| Vertical Reinforcing | 25 M @ 300 | 25 M @ 225 | Min. Reinf. Eqn. |
| Transverse Reinforcing | 10 M @ 150 Grid | 15 M @ 225 H x 140 V | Constructability |
| Prestressing | 7 Strands @ 600 | 5 Strands @ 450 | Constructability |

Table 6.6 Changes in the Reinforcement and Prestressing in the Support Wall Design to NPD NS 3473, to Achieve Minimum Reinforcement and Constructability Requirements, from the Design for Strength and Serviceability Requirements Only (dimensions are in mm).

6.4 Summary of Designs

The object of this design study is to determine the reinforcing and prestressing steel weights for designs done to:

- CSA-S474
- NPD/NS 3473

The steel weights for the ice, tie, and support wall were calculated for the cases of:

- strength and serviceability only; and,
- detailing requirements of minimum reinforcement, as well as a practical layout that is constructible.

The steel weights in the design of the three elements are summarized in Table 6.7.

| Item | Element | | | | | |
|--|----------------------------------|-----------------------|----------------------------------|-----------------------|----------------------------------|-----------------------|
| | Ice Wall | | Tie Wall | | Support Wall | |
| | Strength and Serviceability Only | Detailing Included | Strength and Serviceability Only | Detailing Included | Strength and Serviceability Only | Detailing Included |
| Design to CSA-S474 | 176 kg/m ³ | 231 kg/m ³ | 279 kg/m ³ | 322 kg/m ³ | 305 kg/m ³ | 358 kg/m ³ |
| Design to NPD/NS 3473 | 205 kg/m ³ | 243 kg/m ³ | 273 kg/m ³ | 306 kg/m ³ | 292 kg/m ³ | 322 kg/m ³ |
| % CSA Steel Weight is heavier than NPD | -16% | -5% | 2% | 5% | 4% | 11% |

Table 6.7 Summary of Reinforcing and Prestressing Steel Weights per m³ of Concrete

The differences in steel weight result from the reasons given in the following sections.

6.4.1 Ice Wall

The governing load conditions for the two codes are similar, ie. the 1 in 100 year iceberg, but with the following differences:

- The CSA loading is for a control of local damage criterion. This is a SLS condition where the reinforcing steel stress, which governs over the concrete stress limitation, is not allowed to exceed 0.9 fy. The load factor is 1.0 as are the material resistance factors.
- The NPD/NS 3473 loading is ULS with a load factor of 1.3. The materials factors are 0.8 for concrete and 0.87 for reinforcing steel.

There is 16% less steel in the CSA ice wall after the strength design, but only 5% less steel when the minimum reinforcement requirements are applied.

As noted in Section 6.1 above, a ULS run on the 1 in 100 year iceberg (not required by the CSA code) shows only a load factor of 1.1 where 1.35 is usually applied to a frequent environmental load. As a result, it is recommended that consideration be given to increasing the return period of the local damage control criterion.

The transverse steel requirements for the ice wall were calculated using two completely different methods for the two codes. The simplified method used in NS 3473 required a steel area of 5,000 mm²/m², while the use of SHELL474 required a steel area of 4,450 mm²/m² for the CSA ice wall. It should also be noted that the NS approach was for a ULS load combination using the 1 in 100 year iceberg, while the CSA approach was for a local damage load combination for the 1 in 100 year iceberg.

6.4.2 Tie Wall

The difference in steel weight is only 2% after designing for strength and serviceability, with the CSA design requiring more steel. The governing load conditions for the two codes are both for the SLS crack control condition, but with the following differences:

- The CSA loading is for an annual wave, which is 77% of the 1 in 100 year wave at Hibernia.
- The NPD/NS 3473 is for the wave that is exceeded 100 times in 18 years, which is 74% of the 1 in 100 year wave at Hibernia.

For the tie and support walls, the cracks open up, under tension, across the full width of the section. Under this condition, the cracking equations become:

- For CSA:

$$w_{avg} = \epsilon_1 S_m = \epsilon_1 [2(c + .1s) + 0.1 d'_{bc} h_{cf} b / A_s]$$

where: w_{avg} = average crack width at the surface, limited to 0.3 mm in this case

S_m = average crack spacing

ϵ_1 = principal strain at the surface

c = the concrete cover = 55 mm in this case

s = bar spacing = 190 mm (or 150 mm, depending on the face)

d'_{bc} = effective bar diameter = 35 mm in this case

h_{cf} = 275 mm (1/2 the wall thickness, in this case)

b = width of the section, 1,000 mm

A_s = 5,263 mm²/m

$$\text{so, } w_{avg} = \epsilon_1 [2(55 + .1 \times 190) + .1 \times 35 \times 275 \times 1,000 / 5,263] = 331 \epsilon_1$$

- For NPD:

$$\begin{aligned} w_k &= 1.7 r \epsilon_1 S_m \\ S_m &= 2(c + 0.1s) + k_1 k_2 \phi / \rho_r \end{aligned}$$

where: w_k = characteristic crack width at the level of the reinforcement, limited to 0.5 mm in this case

$$\begin{aligned}
r &= \text{ratio in Clause 15.6.2 of NS 3473, approximately 0.9 in this case} \\
\epsilon_1 &= \text{principal strain at the level of the reinforcement} \\
S_{mm} &= \text{mean crack spacing at the level of the reinforcement} \\
k_1 &= 0.4 \text{ for ribbed bars} \\
k_2 &= 0.25 (\epsilon_1 + \epsilon_2)/\epsilon_2 = 0.25 \text{ for a uniform crack} \\
c &= \text{cover} = 55 \text{ mm in this case, using the CSA design} \\
s &= \text{bar spacing} = 190 \text{ mm} \\
\phi &= 35 \text{ mm} \\
\rho_r &= A_s/A_c = A_s/bh_{cf} \\
h_{cf} &= 275 \text{ mm as in CSA above} \\
b &= 1,000 \text{ mm} \\
A_s &= 5263 \text{ mm}^2/\text{m as in CSA above} \\
\rho_r &= 0.19 \text{ in this case} \\
\text{so, } s_k &= \epsilon_1 1.7 \times .9 [2(55 + .1 \times 190) + .4 \times .25 \times 35/.019] \\
&= 508\epsilon_1
\end{aligned}$$

The following relative crack width ratio for the two codes is obtained, where the calculations are made using the simplifying assumption of a constant crack width through the thickness of the wall:

$$\frac{w_k}{w_{avg}} = \frac{508}{331} = 1.53$$

The allowed crack width ratio is:

$$\frac{0.5 \text{ mm}}{0.3 \text{ mm}} = 1.67$$

As can be seen, there is very little difference between the two codes for a through crack condition of constant thickness. More steel is required in the CSA code as the calculated relative crack width ratio of NPD to CSA for this example is 9% lower than the allowed crack width ratio of NPD to CSA.

After the minimum reinforcement provisions are required, and adjusting the arrangement to make it practical for construction, the CSA design has 5% more steel than the NPD design.

6.4.3 Support Wall

The difference in steel weight is only 4% after designing for strength and serviceability only, with the CSA design requiring more steel. The difference increases to 11% after incorporating the minimum reinforcement provisions and adjusting the arrangement to make it practical for construction, with the CSA design again requiring more steel.

The crack width comparison between the two codes is basically the same as for the tie wall, except that the wall thickness is greater so that the h_{ef} calculation is not limited by half the wall thickness, which introduces subtle changes in the crack width calculation.

As the minimum reinforcement requirements make a significant difference in this case, the differences in this aspect of the codes are noted below:

Minimum Reinforcement Required by CSA

a. X-Reinforcement (Horizontal, in Plane Reinforcement)

- 35 M bars spaced at 135 mm for a steel area of $A_s = 7,407 \text{ mm}^2/\text{m}$ near each face
- $.003 A_c = 3,100 \text{ mm}^2/\text{m}$
- $A_s = (f_{cr} + w) b h_{ef} / f_y$
- $h_{ef} = 55 + 35 + 7.5 \times 35 = 352.5$
- $A_s = 3,375 \text{ mm}^2/\text{m}$

In this case, no change is required due to the minimum reinforcement requirements.

b. Y-Reinforcement Required by CSA

- 25 M bars spaced at 300 mm for a steel area of $1,667 \text{ mm}^2/\text{m}$ near each face.
- $.003 A_c = .003 \times 700 \times 1,000 = 2,100 \text{ mm}^2/\text{m}$
- $A_s = (f_{cr} + w) b h_{ef} / f_y$

where:

| | | |
|----------|---|---|
| f_{cr} | = | 2.83 MPa |
| w | = | 1 MPa approximately |
| b | = | 1,000 mm |
| h_{ef} | = | $55 + 35 + 25 + 7.5 \times 25 = 302.5 \text{ mm}$ |
| f_y | = | 400 MPa |

so, $A_s = 2,896 \text{ mm}^2/\text{m}$

In this case the equation governs, and $2,896 \text{ mm}^2/\text{m}$ is required, which is achieved by using 35 M bars spaced at 300 mm (using 25 M bars at 150 mm would work as well).

Minimum Reinforcement Required by NPD/NS 3473

a. X-Reinforcement

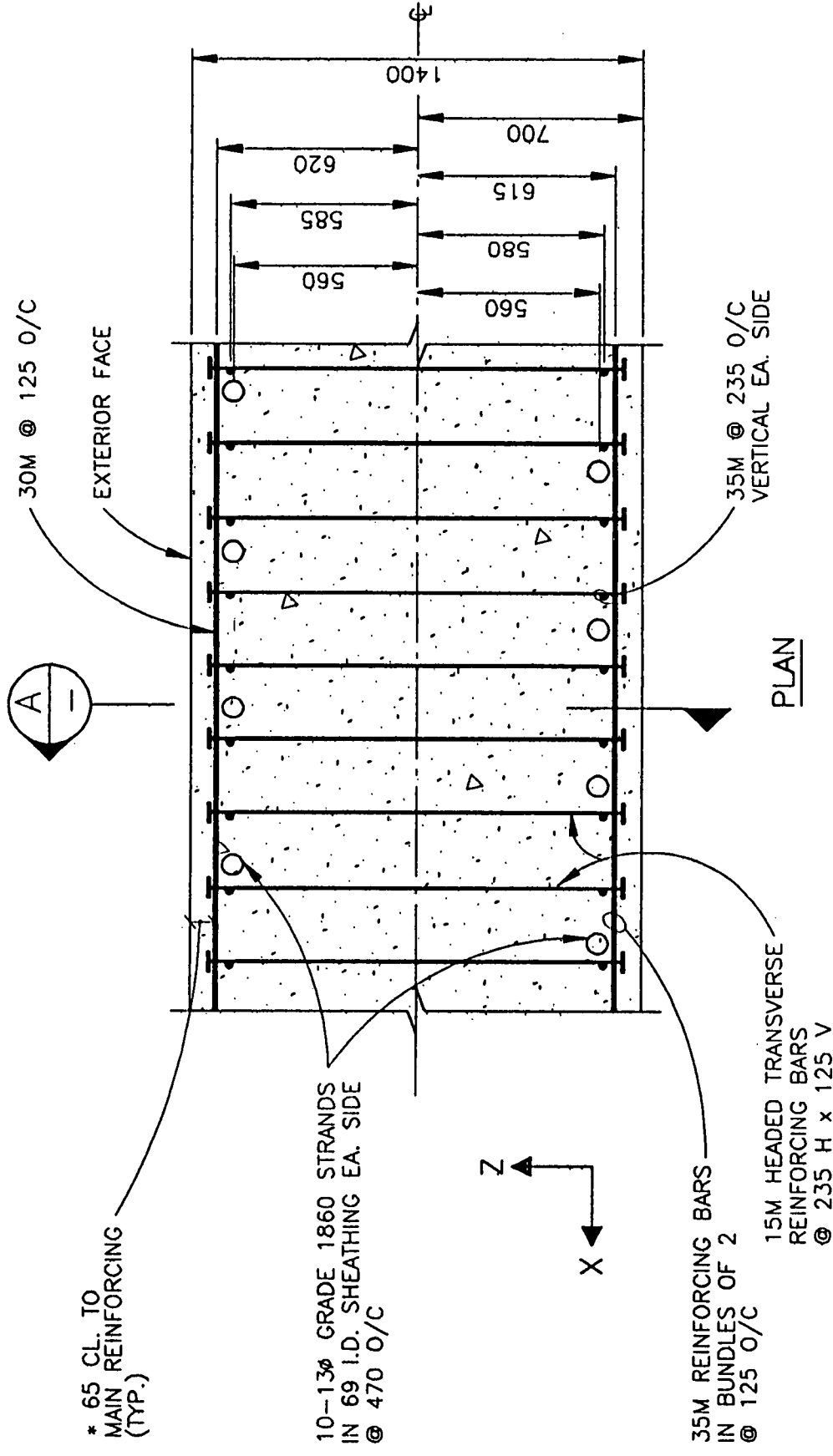
- 35 M bars spaced at 140 mm for a steel area of 7,143 mm²/m near each face
- $$\begin{aligned} A_s &= .28A_c f_{tk} / f_{sk} \\ &= .28 \times 700 \times 1,000 \times 3.51 / 400 \\ &= 1,720 \text{ mm}^2/\text{m} \end{aligned}$$

In this case, no change is required.

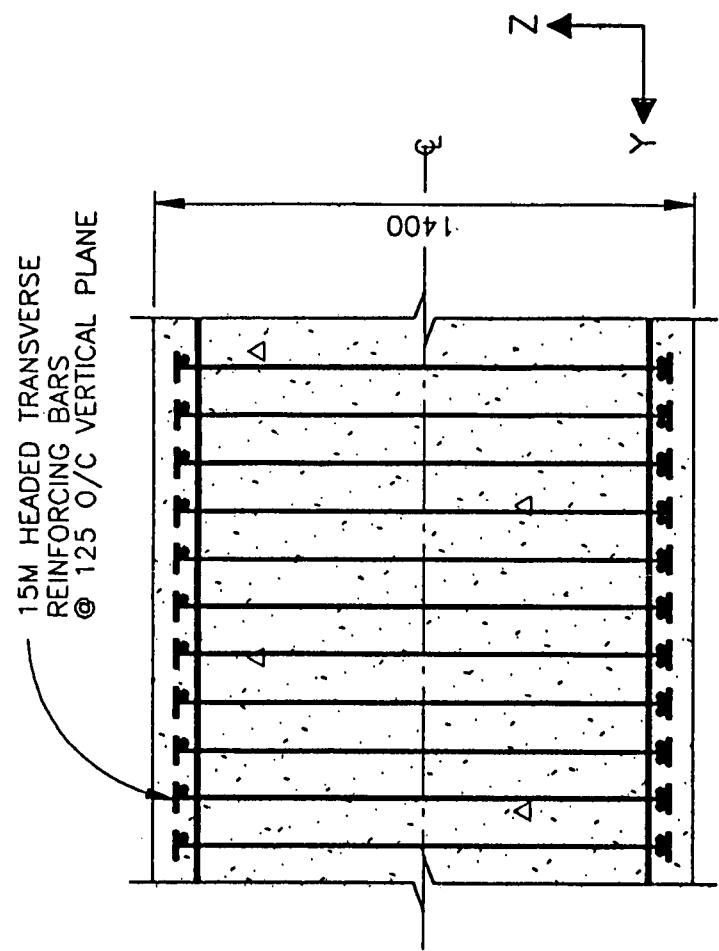
b. Y-Reinforcement

- 25 M bars at 300 mm for a steel area of 1,667 mm²/m near each face
- $$\begin{aligned} A_s &= 0.28A_c f_{tk} / f_{sk} \\ &= .28 \times 700 \times 1,000 \times 3.51 / 400 \\ &= 1,720 \text{ mm}^2/\text{m} \end{aligned}$$

In this case, additional steel area is required and is achieved by arranging the y-reinforcement as 25 M bars at a spacing of 225 mm.



PLAN



ELEVATION

SECTION A

* NOTE:
IN ACCORDANCE WITH CLAUSE
5.4.1 OF S474 - M1989

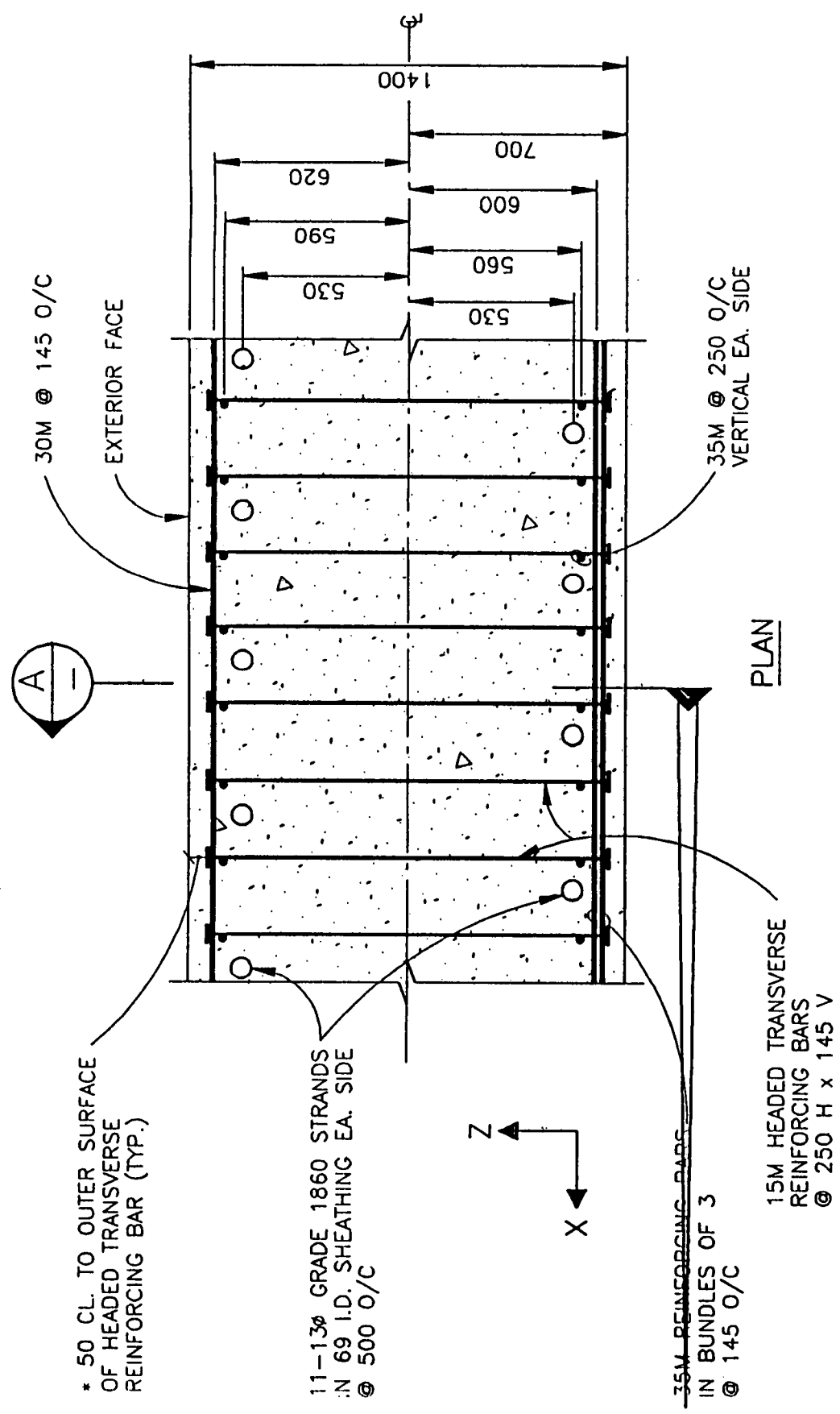
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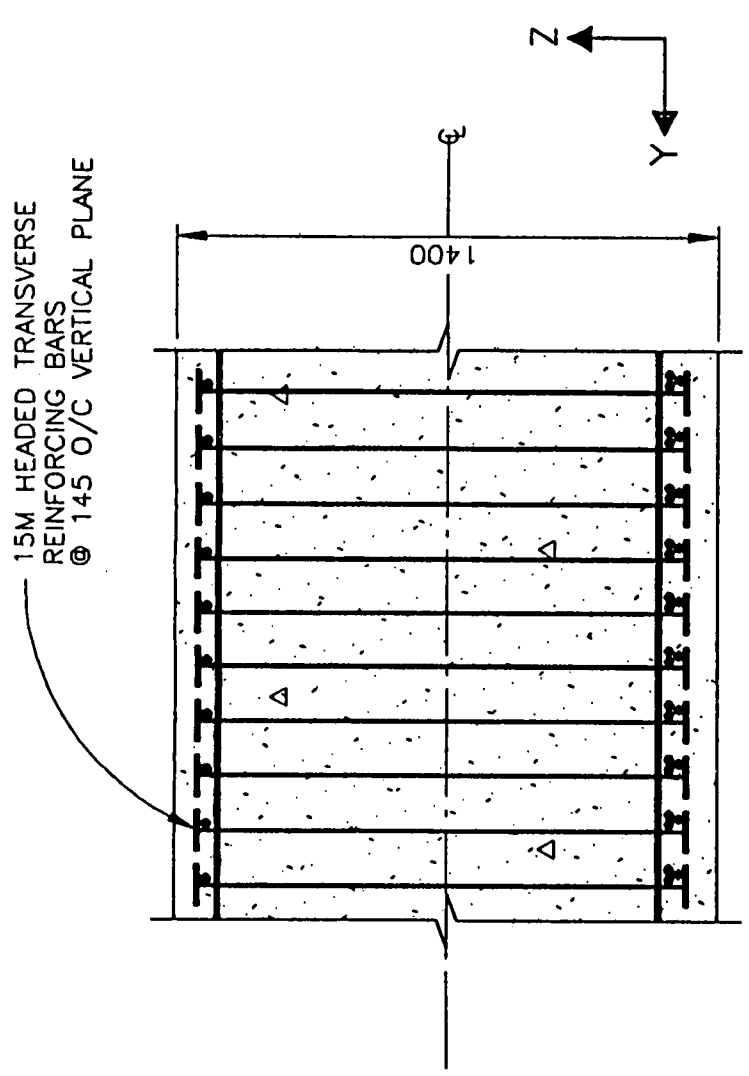
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| Project | TYPICAL ICEWALL SECTION CSA DESIGN | | | | |
| Scale | 1:20 | Design | SY | Drawn | RK |
| Date | SEPT 3/91 | Check | NA | Approved | NA |
| DWG. NO. | 91624 | - FIG. 6.1 | | REV. | |



* NOTE:
IN ACCORDANCE WITH CLAUSE
17.1.8 OF NS 3473 3RD EDITION



PRELIMINARY

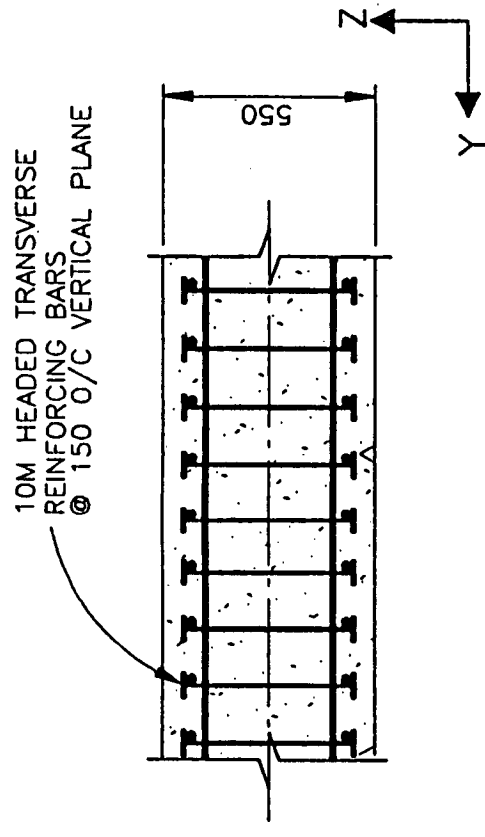
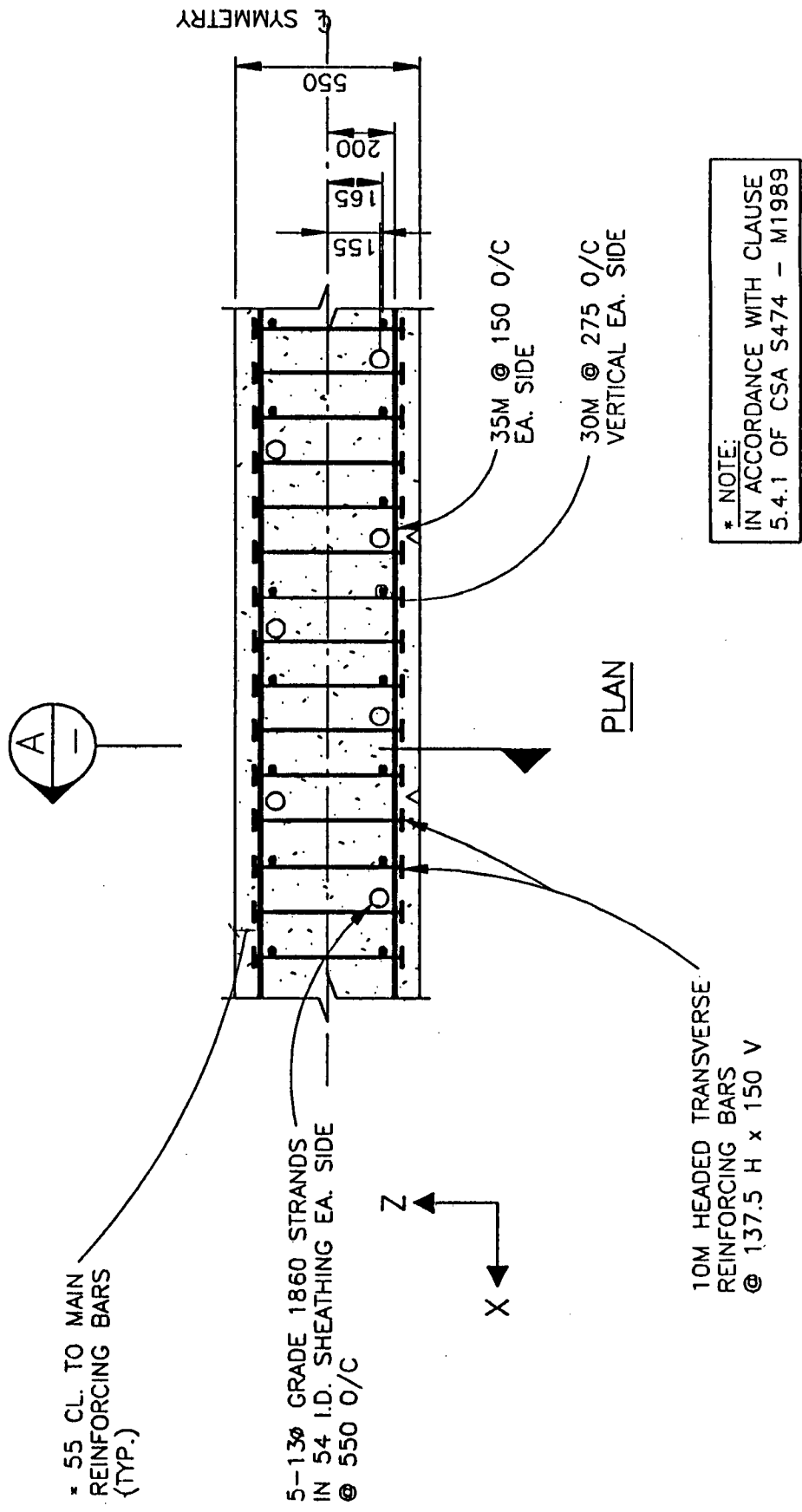
Westmar Consultants Inc.
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Project **TYPICAL ICEWALL SECTION NPD DESIGN**

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|-------|-----------|--------|----|----------|----|
| Scale | 1:20 | Design | SY | Drawn | RK |
| Date | SEPT 3/91 | Check | NA | Approved | NA |

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| DWG. NO. | 91624 | FIG. 6.2 | REV. |
|----------|-------|----------|------|

| REV | DATE | REVISION | BY | CKD | Client |
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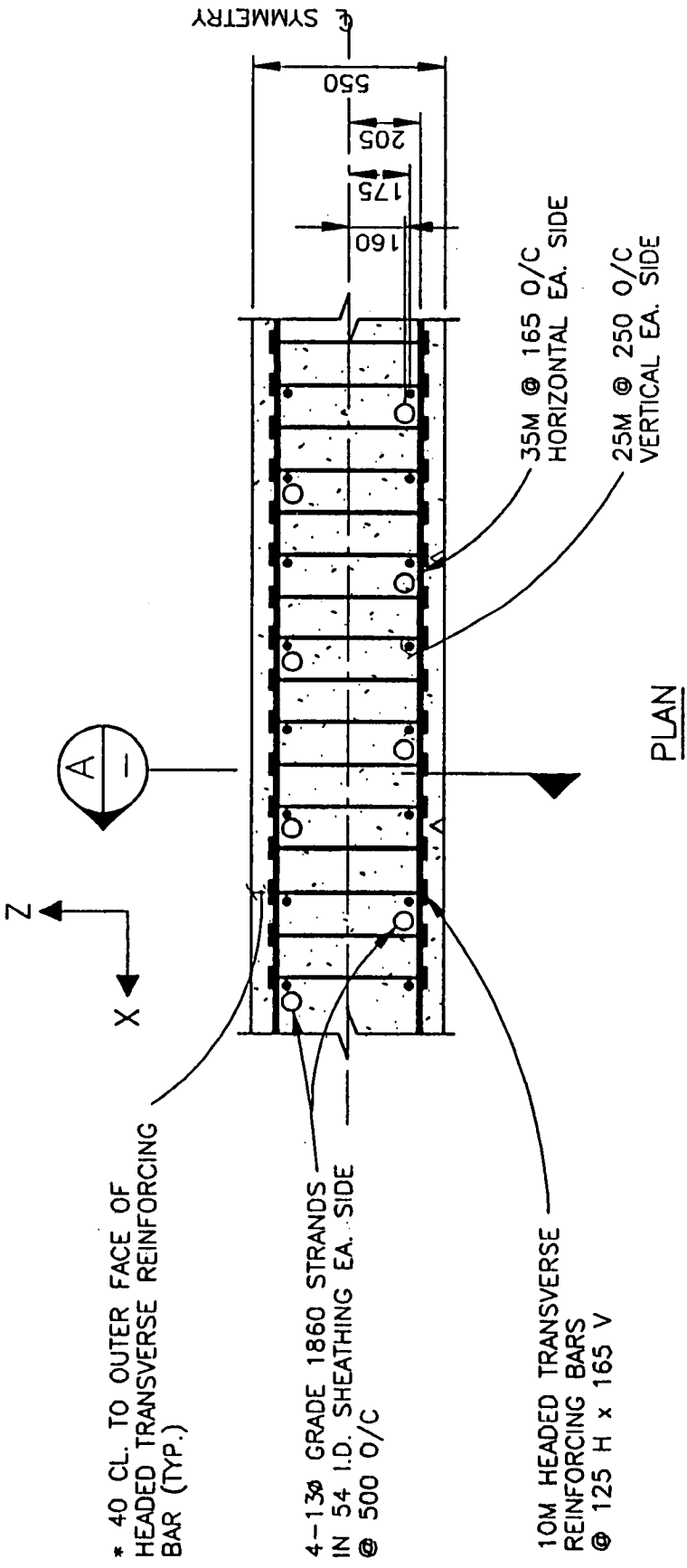
PRELIMINARY

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| | | | |
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| Project TYPICAL TIE WALL SECTION CSA DESIGN | | | |
| Scale | 1:20 | Design | SY |
| Date | SEPT 3/91 | Check | NA |
| Drawn | | Approved | NA |
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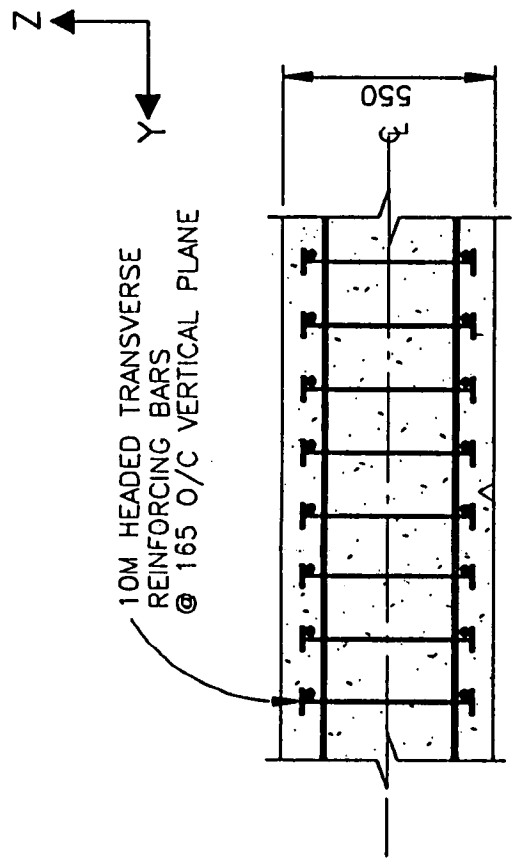
DWG. NO. 91624 - FIG. 6.3 REV.

| REV | DATE | REVISION | BY | CKD |
|-----|------|----------|----|-----|
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* NOTE: IN ACCORDANCE WITH CLAUSE 17.1.8 OF NS 3473 3RD EDITION

PRELIMINARY



SECTION A

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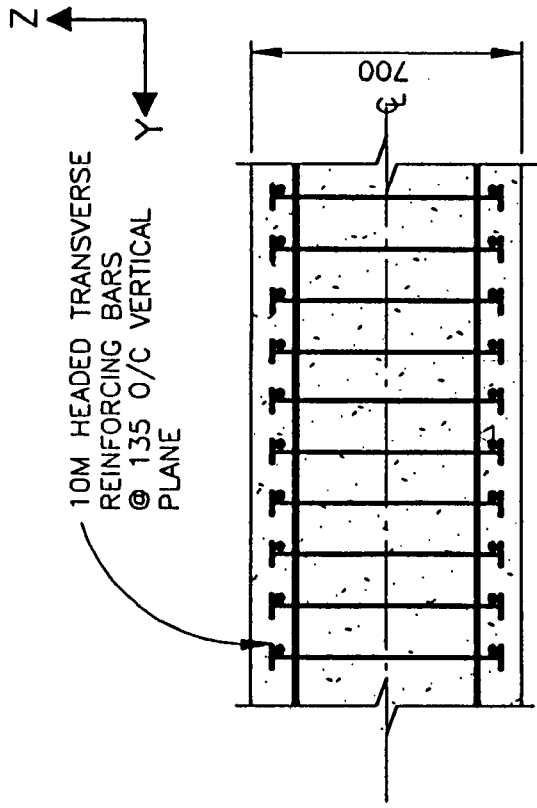
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| Date | SEPT 3/91 | Check | NA | Approved | NA |

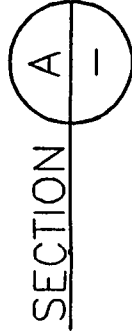
DWG. NO. 91624 - FIG. 6.4 REV.

| REV | DATE | REVISION | BY | CKD |
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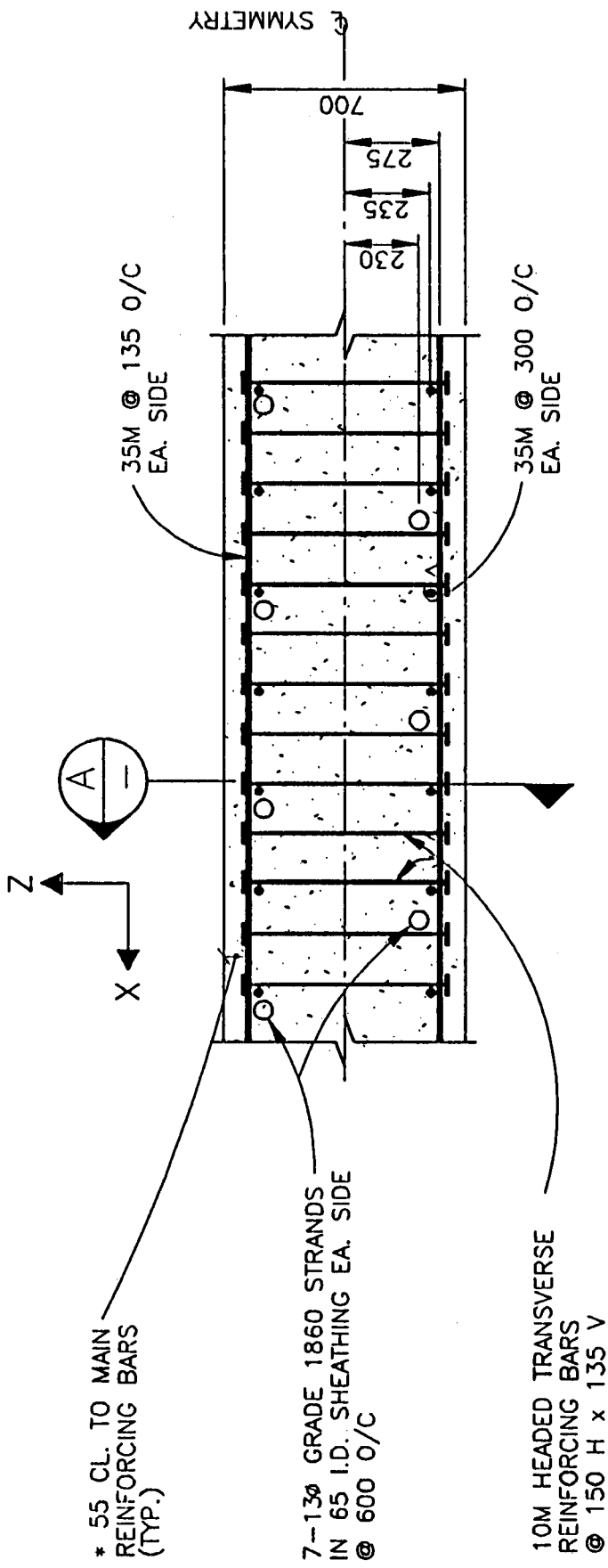
Client



ELEVATION



SECTION A



PLAN

* NOTE:
IN ACCORDANCE WITH CLAUSE
5.4.1 OF CSA S474 - M1989

PRELIMINARY

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Project

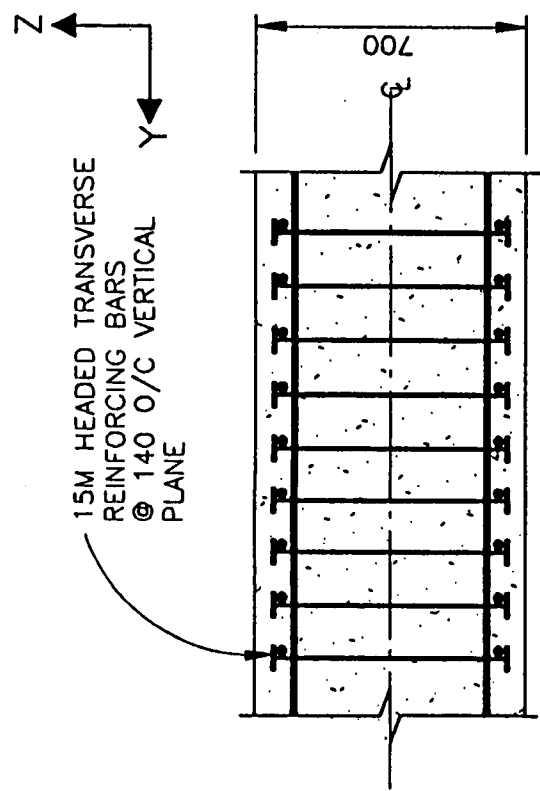
TYPICAL SUPPORT WALL SECTION CSA DESIGN

| | | | | | |
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| Date | SEPT 3/91 | Check | NA | Approved | NA |

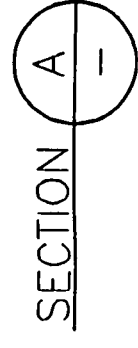
DWG. NO. 91624 - FIG. 6.5 REV.

Client

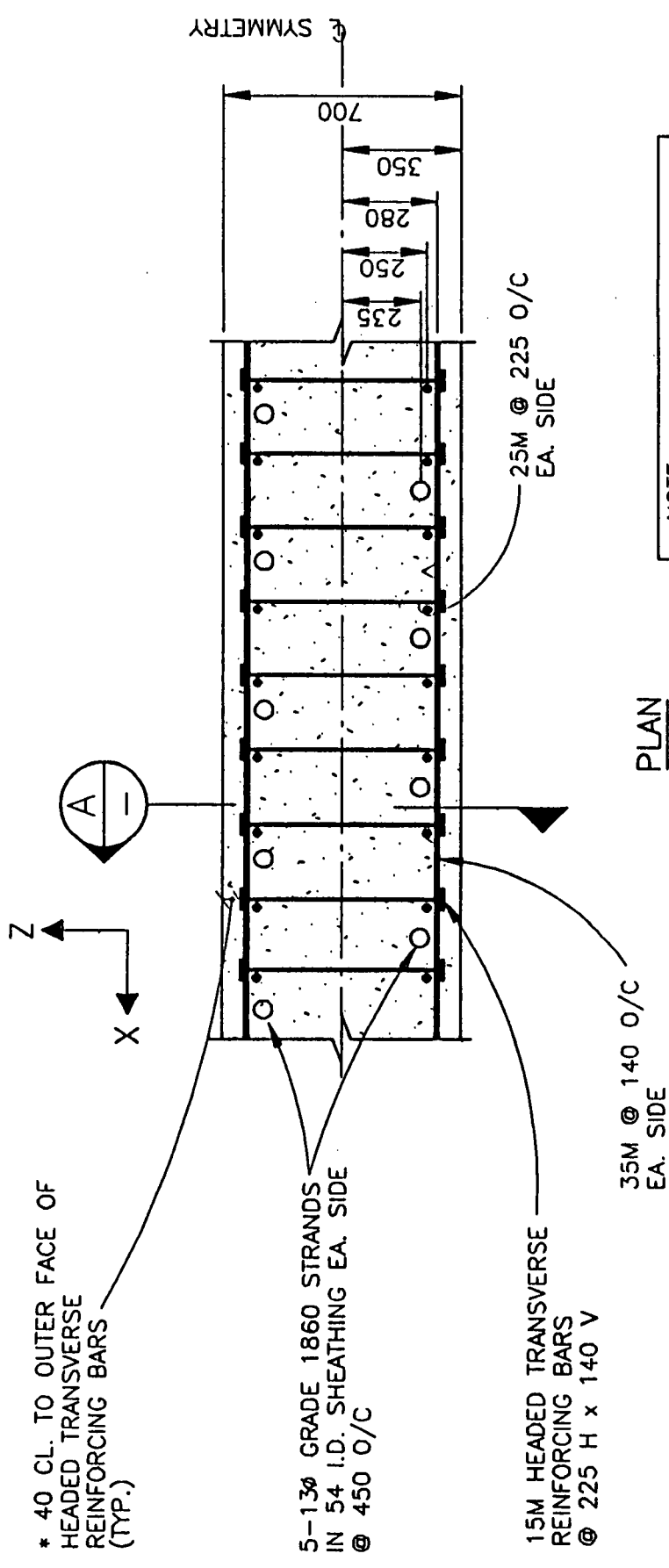
| REV | DATE | REVISION | BY | CKD |
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ELEVATION



SECTION A



PLAN

* NOTE:
IN ACCORDANCE WITH CLAUSE
17.1.8 OF NS 3473 3RD EDITION

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| | | | |
|---|-----------|----------|----|
| Project | | | |
| TYPICAL SUPPORT WALL SECTION NPD DESIGN | | | |
| Scale | 1:20 | Design | SY |
| Date | SEPT 3/91 | Check | NA |
| DWG. NO. 91624 - FIG. 6.6 | | Drawn | RK |
| | | Approved | NA |
| | | REV. | |

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| Client | |
| REVISION | BY CKD |
| DATE | |

7.0 FATIGUE CHECKS

Fatigue checks of the wall designs for the two codes were made using the reinforcing schemes incorporating strength and serviceability requirements only, as reported in the following sections.

7.1 Ice Wall

The stress ranges for the CSA and NPD/NS 3473 designed ice walls, as determined by the SHELL474 runs, are all lower than is required to make a Miner's rule summation, as given in Table 7.1.

| Item | Design Code | |
|----------------------------|-------------|-------------|
| | CSA | NPD/NS 3473 |
| Reinforcement Stress Range | 125.3 MPa | 122.2 MPa |
| Prestressing Stress Range | 3.5 MPa | 3.4 MPa |
| Concrete Stress Range | 0.7 MPa | 0.7 MPa |

Table 7.1 Stress Ranges in the Ice Walls Due to the Fatigue Load Combination

As these stress ranges are less than the limits for checking fatigue, as given in Section 4.5, the design of the ice wall can be considered adequate for wave induced stress cycles without performing detailed fatigue damage calculation.

7.2 Tie Wall

The stress ranges in the CSA and NPD/NS 3473 designed tie walls, as determined by the SHELL474 runs, are given in Table 7.2.

| Item | Design Code | |
|----------------------------|---------------------------------------|---------------------------------------|
| | CSA | NPD/NS 3473 |
| Reinforcement Stress Range | 228 MPa | 248.7 MPa |
| Prestressing Stress Range | 5.9 MPa | 6.6 MPa |
| Concrete Stress Range | 1.11 MPa (compression/compression) | 1.03 MPa (compression/compression) |

Table 7.2 Stress Ranges in the Tie Walls Due to the Fatigue Load Combination

The fatigue life of the ordinary reinforcement is checked using the S-N curve given in Clause 13.2.2 of NS 3473:

$$\log N = 19.6 - 6 \log S$$

It can be shown that the CSA method, where the stress range is divided by 1.5, calculates 14% more damage to a given design and in the same wave climate, than the NPD method, where the number of cycles is multiplied by 10.

The following Miner's rule summations were calculated for the ordinary reinforcement, as extracted from the detailed calculations:

- For the CSA Tie Wall:

$$\sum \frac{N_i}{N_{fi}} = .393$$

- For the NPD Tie Wall:

$$\sum \frac{N_i}{N_{fi}} = .582$$

The fatigue checks of the tie wall show that the design is adequate for wave induced stress cycles.

7.3 Support Wall

The stress ranges in the CSA and NPD/NS 3473 designed support walls, as determined by the SHELL474 runs, are given in Table 7.3.

| Item | Design Code | |
|----------------------------|--------------------------------------|---------------------------------------|
| | CSA | NPD/NS 3473 |
| Reinforcement Stress Range | 188.8 MPa | 194.5 MPa |
| Prestressing Stress Range | 3.6 MPa | 3.9 MPa |
| Concrete Stress Range | 0.2 MPa (compression/compression) | 0.21 MPa (compression/compression) |

Table 7.3 Stress Ranges in the Support Walls Due to the Fatigue Load Combination

The fatigue life of the reinforcing steel was checked using the S-N curve in Clause 13.2.2 of NS 3473, and as given in Section 7.2 above.

The following Miner's rule summations were calculated for the ordinary reinforcement, as extracted from the detailed calculations:

- For the CSA Support Wall:

$$\sum \frac{N_i}{N_{fi}} = .127$$

- For the NPD Support Wall:

$$\sum \frac{N_i}{N_{fi}} = .133$$

The fatigue checks of the support wall show that the design is adequate for wave induced stress cycles.

8.0 DUCTILITY CHECKS

Ductility check calculations on the CSA wall designs were made using the reinforcing schemes incorporating strength and serviceability requirements only.

The object of these checks is to determine if the wall elements fail in a ductile manner, thus enabling the structure to absorb considerable amounts of energy under a 1 in 10,000 year iceberg loading condition. The ductility criteria is addressed in the two codes with either a specific ductility curve (CSA) or a limit state of progressive collapse where an extreme, unusual or accidental load on the ice wall does not cause an extensive collapse (NS 3473).

Critical elements in ductility checks are elements subjected to high compressive loads. The ductility of an element subjected to tension and bending type loads easily satisfy CSA Clause 8.4.4. The same is not true for compressive type loadings.

Transverse reinforcement is used to increase the ductility of compression elements by confining the concrete; this enables the concrete to carry significant loads at large strains. The stress-strain curves in the CSA and NPD/NS 3473 codes are for unconfined concrete. Compressive failures using these curves are not ductile, and the calculations show that walls designed with these stress-strain curves do not meet the ductility requirements of CSA Clause 8.4.4.

Using the Kent and Park stress-strain curves, which accounts for increased ductility with increasing quantities of transverse reinforcement, it was found that the ductility requirements of CSA were satisfied. The calculation results using both the unconfined and confined concrete stress-strain curves are summarized in the following sections.

8.1 Ice Wall

To meet the ductility requirements of Clause 8.4.4 of S474 in a crushing mode of failure, .075 % transverse reinforcement is required, using the Park and Kent curve. As the CSA and NPD designs have .445 % and .5 % transverse reinforcement, respectively, they will fail in a ductile manner meeting the CSA conditions.

The SHELL474 runs demonstrate the insufficient ductility when the unconfined stress-strain curve is used, and the more than adequate ductility when the confined stress-strain curve is used.

8.2 Tie Wall

To meet the ductility requirements of Clause 8.4.4 of S474 in a crushing mode of failure, .37 % transverse reinforcement is required, using the Park and Kent curve. As the CSA and NPD designs both have .445 % transverse reinforcement, they will fail in a ductile manner meeting the CSA conditions.

The SHELL474 runs demonstrate the insufficient ductility when the unconfined stress-strain curve is used, and the more than adequate ductility when the confined stress-strain curve is used.

8.3 Support Wall

To meet the ductility requirements of Clause 8.4.4 of S474 in a crushing mode of failure, .37% transverse reinforcement is required, using the Park and Kent curve. As the CSA and NPD designs both have .445% transverse reinforcement, they will fail in a ductile manner meeting the CSA conditions.

The SHELL474 runs, demonstrate the insufficient ductility when the unconfined stress-strain curve is used, and the more than adequate ductility when the confined stress-strain curve is used.

8.4 Energy Dissipation

The energy dissipated in deforming the elements that fail in a crushing mode to the 50% f'_c level on the post peak side of the stress-strain curve, is summarized in Table 8.1.

| Item | Energy Dissipated | |
|--------------|--------------------------------|------------------------------|
| | Unconfined Stress-Strain Curve | Confined Stress-Strain Curve |
| Ice Wall | 18 MJ | 85 MJ |
| Tie Wall | N/A | N/A |
| Support Wall | 27 MJ | 75 MJ |

Table 8.1 Energy Dissipated up to the 50% f'_c Post Peak Deformation as Governed by the Elements Failing in Crushing

The total kinetic energy of a 1 in 10,000 year iceberg is in the order of 2,000 MJ. The energies absorbed in the initial part of the failure curve of a concrete element is only a small part of the total energy dissipated. Much greater amounts of energy are dissipated in the "tail" of the ductile stress-strain curve, as shown below:

Ice Wall

- Assume a 26 m high load patch and failure height
- Assume a 3 m crushing length at $0.2 f'_c = 10 \text{ MPa}$ until about 50% of a tooth is crushed
- Energy Dissipated = $1.2 \text{ m} \times 26 \text{ m} \times 10 \text{ MPa} \times 3 \text{ m} = 936 \text{ MJ}$

Support Wall

- Assume a 26 m high load patch and failure height
- Assume a 3.7 m crushing length, until oil storage is ruptured, at $0.2 f'_c = 10 \text{ MPa}$
- Energy Dissipated = $.55 \text{ m} \times 26 \text{ m} \times 10 \text{ MPa} \times 3.7 \text{ m} = 529 \text{ MJ}$

As can be seen by the above simplistic calculations, large amounts of energy are dissipated in failing elements properly detailed to fail in a ductile fashion. As 50% of the kinetic energy will be dissipated in crushing the ice, the iceberg will be stopped by the CSA or NPD designed structures presented herein, before human life or the environment are put at risk.

Note that the tie wall is required to maintain the integrity of the structure, and so must also be detailed to be ductile and undergo large strains.

9.0 APPRAISAL OF CSA CODE

The appraisal of the CSA code addresses the following points:

- Overview
- Technical
- Allowance for incorporating new information
- Practicality for design of fixed offshore production structures
- Flexibility for creative design

9.1 Overview

The set of codes examined herein, namely CSA S471 and S474-M1989, demonstrate a high level of sophistication, particularly when applied to the ice laden waters of the Canadian Offshore.

The two computer codes developed in earlier verification projects, namely the DnV Probabilistic Framework for calculating global ice loads and SHELL474 for analyzing elements of the structure, add a level of sophistication to any design process that no other code writing bodies have attempted.

The ULS analysis procedures were verified and found to be straight forward in incorporating into the design process, with the exception that a new load combination number 10 is required.

The provisions on ductility appear appropriate in that the designer must incorporate confining steel to the various wall elements to ensure that the elements fail in a ductile fashion in a crushing mode of failure. This in turn allows considerable amounts of energy to be absorbed during indentation by a 1 in 10,000 iceberg, such that the iceberg will be stopped before human life or the environment are put at risk.

A new SLS load combination number 10 should be considered in Table 6.2 of S471, for the local damage control criteria for a specified rare environmental load, to be consistent with Clause 9.6.1 of S474.

The fatigue provisions in CSA were verified, and, for the case of the fatigue life of reinforcing steel acted on by waves over the 18 year life of the structure, were found to be conservative by 14% when compared with the approach used in NPD. More guidance could, however, be provided in the commentary to S474 on the appropriate S-N curves to use; in this study, the ones put forward in NS 3473 were used.

9.2 Allowance for Incorporating New Information

The codes are well organized and contain sufficient subdivision of the material to generally allow incorporation of new information without undue reorganization of the documents. However, the overall organization of S474 requires that the user make numerous cross references to S471 for additional definition. A caution is thereby raised that the authors of each document pay specific attention to document revisions and the related effects resulting in cross referenced documents.

9.3 Practicality for the Design of Fixed Offshore Production Structures

S471 and S474 appear to be useful for design of a specific range of offshore structure types; namely, gravity structures, fixed to the seafloor and producing hydrocarbons in hostile marine environments. The S474 code appears to treat many complex design issues comprehensively, yet practical design formulations are likely to result for the specific class of intended structures; namely, fixed offshore concrete production structures.

9.4 Flexibility for Creative Design

The flexibility of CSA S474 as a design code is greatly enhanced by the "user-friendly" organization of SHELL474. SHELL474 offers rapid analysis of complex loadings and resulting computations of service performance and strength capacity. The thoroughness of SHELL474 affords the designer the opportunity to rapidly develop design solutions for structures having complex geometric arrangements and in doing so, provides an enhanced level of design flexibility when compared to existing codes.

The concept of allowing Safety Class 1 and Safety Class 2 structural elements, as defined by both S471 and S474, also adds flexibility to design formulations. Less critical structural members can be classified as Safety Class 2 elements and this should result in the opportunity to make the overall structure design more efficient and most cost effective.

The code also appears to allow the designer to class the structure differently for different environmental conditions, such as:

- Safety Class 1 for earthquakes
- Safety Class 2 for ice islands

which will allow for more cost effective structures.

10.0 RECOMMENDATIONS

The following items are recommended for consideration by the Code committee as changes to the Code or further studies as identified in verification projects G-2A and G-2B:

1. Control of Local Damage

The "Control of Local Damage" provisions in S474 require the structure to remain elastic under loads that have an annual probability of exceedance of 0.01 (Section 9.6 of S474). However, it was found that when the ice wall is designed to this provision, and it meets all the other provisions, including detailing, in S474, it still only has a calculated load factor of 1.11 against the ULS or ultimate limit state (the start of failure) of the ice wall, where the load factor on this event is only 1.0 (a normal ULS load factors is 1.35; the ULS of a rare environmental event with a return period of 100 years is not a required design condition, as can be seen on Table 6.2 of S471).

A risk/reliability study is recommended to determine the return period for the local damage criterion such that an acceptable level of risk of damage is maintained.

The study should account for:

- a) The probability of exceedance of the load level for specifying local damage control.
- b) The probability of damage to the structure, given the materials factors included in the ULS analysis, integrated over the iceberg loads of longer return period.
- c) A level of risk that the owner is willing to take, consistent with the levels used in developing the code to date.
- d) The incremental cost of increasing the structure reliability versus the cost of repairing the damage should be included as well. It should be noted that total structural failure is not accompanied by the failure of a part of the ice wall due to ductility and redundancy, and this should be accounted for as well.

The control of local damage criterion in S474 governs in the design of ice wall and so the S474 code committee may wish to resolve this question before issuing S474 in final form.

2. Crack Width from Waves

It is felt that the wave loading for the crack width criteria in S474 are overly conservative. At the present time an annual wave is specified in S474, whereas the latest codes in Norway specify the wave that is exceeded 100 times in the life of the structure. For the Hibernia GBS structure with an operating life of 18 years, the Norwegian criteria is for the wave that is exceeded on the average about every 2 months.

A study should be undertaken to examine the strength reduction caused by cracks opening varying amounts and frequency over the operating life of the structure. A probabilistic analysis based on a realistic wave energy spectrum and on the actual observed wave climate at Hibernia, should be undertaken.

Note that the tie wall and support wall, immediately behind the ice wall of the Hibernia GBS 1986 update design, are governed by the crack width criteria in S474.

3. Combinations of Waves with Icebergs

Among the recommendations made in the G-2A report, the examination of the wave event to be combined with varying return period iceberg events remains to be addressed.

A probabilistic study of the combined events and the force levels as well as duration of the force levels should be addressed. The question should be addressed of whether multiple impacts on a damaged ice wall can occur, from the same iceberg, in a given storm. As wind and wind driven currents are part of the driving force on the iceberg, and as the wind also produces storm generated waves, the combined dependency of wind, waves and iceberg velocity should be accounted for. The forces calculated using diffraction theory for the combined iceberg velocity should be accounted for. The forces calculated using diffraction theory for the combined iceberg and structure should be incorporated in the analysis, along with the forces transmitted to the structure from the iceberg as it decelerates and is simultaneously impacted by a wave.