Rapid erection of a concentrate loading facility using precast concrete in a remote location

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Volume 10 • Number 3 • 1983

Pages 456–465
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Received November 12, 1982
Revised manuscript accepted April 18, 1983

This paper presents the design concept and construction details of a concentrate loading facility in northern Manitoba under adverse conditions. The facility was constructed using plant-produced structural components which had to be shipped by rail to the site and assembled using the post-tensioning technique to provide structural integrity.

The main elements of the structure were precast footings, precast retaining walls, and precast, prestressed channel-shaped girders. The project described in the paper is an efficient design highlighted by quality control and significant saving in construction time. The concept lends itself to many types of loading and storage facilities for remote locations.

Cet article traite de la conception, du calcul et de la construction d’une aire de chargement dans le Manitoba septentrional dans des conditions difficiles. Cette aire de chargement fut construite grâce à des éléments structuraux préfabriqués qui ont été expédiés sur le site par rail et assemblés par postcontraintes afin d’obtenir une seule structure.

Les éléments principaux de la structure étaient : des semelles de fondation préfabriquées, des murs de retenue préfabriqués et des poutres, de section en C, préfabriquées et précontraintes. Le projet décrit dans cet article est une conception caractérisée par un contrôle de la qualité et une économie importante de temps lors de la construction. Ce concept peut être appliqué à tous les genres d’aires de chargement et d’entreposage en des endroits éloignés.

[Introduit par la revue]


Introduction

In mid-May, 1981, Genstar Structures Ltd. was asked to provide a proposal on a concentrate loading facility for Sherritt Gordon Mines Ltd. at Lynn Lake, Manitoba (approximately 1100 km north of Winnipeg). The facility would be used to separate and store different concentrates from the mill prior to loading and shipping to southern markets.

Since Lynn Lake is a relatively remote location, cast-in-place concrete could only be produced at a very high cost, due to the lack of batching facilities and raw materials. A further prerequisite was a very tight time schedule, which required the facility to be completed and operable by July 30, 1981. Genstar Structures Ltd. commissioned Penner and Keeler Partners Limited to design a precast concrete structure which could be manufactured in their Winnipeg plant and then be shipped by rail to the site and assembled. The system chosen comprised precast concrete elements erected in pieces and post-tensioned together by means of high tensile steel bars to provide structural integrity.

Design and drawings began immediately, with production of elements starting on June 10, 1981. The structure was shipped to the site on July 20, 1981, and 6 days later erection was completed and the facility was turned over to the owners—4 days ahead of schedule.

General description and design consideration

The structure was designed to provide a loading facility for concentrates for Sherritt Gordon Mines Ltd., with a storage capacity of approximately 90,000 ft³ (2550 m³). Covering an approximate area of 6000 ft² (558 m²), the main elements of the structure were precast footings, precast retaining walls, and precast, prestressed channel-shaped girders (Fig. 1). The width and capacity of the floor deck were designed to accommodate truck loads up to 67 tons (600 kN) capacity.

All elements were required to be fabricated in Winnipeg and shipped by rail to the site. This constraint set a limit on the size, shape, and weight of each individual component of the structure. The size and weight of each structural element were also limited by the capacity of the crane to be used during construction.

Design considerations and construction details of each element of the structure under discussion are presented and illustrated in detail in the following sections of this paper.

Structural components

1. Footings

Footings consisted of concrete panels normally reinforced with conventional bars of grade 60 (400 MPa). Two different types of footings were used for the struc-
Fig. 1. Plan of loading facility.
ture. The first type was designed to support the main intermediate retaining walls and floor deck, whereas the second type was designed to support the back retaining walls and provide resistance against earth fill. Both types of footings were 5 ft (1520 mm) in width and varied in length from 30 to 36 ft (9120 to 10 970 mm).

End-blockings of the Dywidag high-tensile strength deformed reinforcing bars were embedded in the footings and the bars protruded 4 in. (102 mm) above the surface of the footings as shown in Fig. 2. This protrusion was designed for coupling and extension into the retaining wall to provide structural continuity. Normal design procedures were followed for determining the reinforcement in the foundation and end blocks.

2. Retaining walls

Two types of retaining walls were used in the structure. Main walls were designed to support the floor deck (Fig. 3). Side walls were designed mainly to resist backfill and were perpendicular to the main walls. Both types of walls were 16 ft high (4876 mm), with lengths varying from 22 to 35 ft (6700 to 10 670 mm). Due to limitations imposed by shipping and erecting procedures, all walls were divided horizontally into two panels of approximately equal weight. The main retaining walls were crowned at the top as shown in Fig. 4 to provide sufficient bearing area to support the channel-shaped girders of the floor deck.

In designing the main walls, different loading condi-
ELEVATION

Fig. 3. Main retaining wall.
tions were considered in anticipation of occasions when one storage cell would be fully loaded while others were not in use. This loading condition would induce cases of reverse applied moment on the retaining walls. As a result of this, the Dywidag bars were oriented in staggered locations as shown in the plan of Fig. 5. The retaining wall panels were reinforced with conventional reinforcement bars which were mainly designed to satisfy erection loads.

The height of the two panels for each retaining wall was selected to achieve equality of weight. However, the location of the interface between panels was located to avoid the maximum moment. Steel angles were embedded at the edge of each panel at their interfaces and welded to each other by the use of steel plates to provide a mechanical means to transfer the shear between the two panels. Both panels of the main retaining walls were connected to the side retaining walls by angles provided with slotted holes to allow for the required expansion joints.

At the bottom of each panel, pockets were provided to facilitate placement of the couplers used to extend the Dywidag bars through the walls, as shown in Fig. 6. Holes were formed in the walls during casting of the concrete by using steel pipes which were removed the second day after casting. The hole diameters were 2\(\frac{1}{2}\) in. (64 mm) to allow 1\(\frac{1}{4}\) in. (38 mm) tolerance and thereby facilitate the alignment and levelling of the retaining wall with respect to the footing.

At the crown of the retaining walls, recesses were provided to accommodate the other end-blocking of the Dywidag bars. These recesses were carefully planned to avoid interfacing with the holes provided to support the channel-shaped girders, as shown in Fig. 5. These holes were designed to match another set of holes in the channel girders, through which continuity would be provided by steel bars. The holes were filled with grout after placement of the floor girder (Fig. 4).

3. Floor girders

Floor girders were channel-shaped and were prefabricated and prestressed using 7\(\frac{1}{2}\) in. (13 mm) di-
ameter 7-wire strands as shown in Fig. 7. A total of 18 girders were used to complete the floor deck. The channel-shaped girders were 34 in. (864 mm) deep and 34 ft 11 in. (10,640 mm) in length and were placed in groups of four arranged side by side, crossing the span between retaining walls.

Girders at the outside edge were provided with steel grids to facilitate cleaning of the floor deck during unloading operations. The steel grids were fabricated from angles and steel plates and were embedded in the concrete. Composite action between the concrete and the steel grid was implemented in designing the girder (Fig. 8).

Cross-diaphragms were provided at the mid-span of each girder to facilitate lateral post-tensioning of the four girders to each other to enhance the lateral resistance of the floor. Diaphragms were also provided at both ends of each girder to achieve the same purpose.

Two holes were provided at each end of each girder to match holes provided in the retaining wall crowns. Steel reinforcement bars were used to provide continuity and resistance to horizontal reactions. At one side, holes were filled with high-strength grout. Holes on the outer side were filled with semiductile material to allow for horizontal movements (Fig. 1).

An extra line of girders was provided, completely separate from the floor deck and spanning every other retaining wall, to provide horizontal restraint for the retaining walls and, at the same time, support the catwalk to facilitate unloading operations.

**Construction sequence**

The initial stage in the construction process was the preparation of the site. The soil was compacted and levelled with a 1–2 in. (25–50 mm) sand base and after placement of the foundations the main retaining
NOTE: ALL DIMENSIONS IN BRACKET ARE IN MM.

Fig. 6. Typical bottom detail of the main retaining wall.
FORMING SECTION

SECTION C–C

SECTION B–B

NOTE: ALL DIMENSIONS IN BRACKET ARE IN MM.

Fig. 7. Typical floor girder details.
walls were erected by placing the lower panels, as shown in Fig. 9. Each panel was supported temporarily by steel braces which in turn were connected to a hollow-core panel placed between the foundations. A Dywidag steel bar was threaded into the holes at each extreme end of each lower panel and coupled with the bars which protruded from the foundation at the same locations.

At the same time, bars were extended upwards to a sufficient height to tie in the upper panels of each retaining wall. Shims were used between the lower panels and the foundation, and between the upper and lower panels, to facilitate levelling of the panels and to provide enough space for future dry-packing.

After placement of the upper panels two bars were prestressed to the required level and provided temporary support. Construction was continued by dry-packing the shimmmed spaces and threading all the Dywidag bars through the holes in the retaining walls. All the Dywidag bars were coupled with the matching protrusions from the footing and then post-tensioned from the top to the specified forces by using hydraulic jacks. Special recesses provided in the ends were then filled with high-strength grout to protect the Dywidag end-blockings.

After erection of the main walls, erection of the side retaining walls was undertaken, following the same procedure. Both sets of walls were connected to each other with two different types of joints, carefully designed to ensure sliding freedom to accommodate temperature changes and lateral movements of the walls (Fig. 10).

After completion of the construction of the retaining walls, the floor girders were positioned and the holes at each end of each girder were matched with the holes provided in the retaining wall crowns, as shown in Fig. 11. Reinforcing bars were provided to achieve continuity between the girders and the retaining walls and the holes were grouted. The girders were post-tensioned laterally at each end and at midspan.

The construction process was concluded by posi-
Conclusion

This unique design indicates the capability and potential of precast concrete structures in the North, where conditions are not ideal and construction materials not readily available. The designers feel that the efficiency and quality control achieved by plant-produced items and assembly on-site were the main reasons that the project was successfully completed within very tight time constraints.

Acknowledgements

We are indebted to the precast concrete manufacturer and contractor, Genstar Structures Ltd. (formerly Con-force Products Ltd.), Winnipeg, Manitoba and the owner, Sherritt Gordon Mines Ltd., Lynn Lake, Manitoba.