# Load Tests On Small Diameter Augered Cast-In-Place Piles Through Fill

**W. Ken Beck, P.E., M.ASCE[1] and Patrick J. Harrison, P.E., M.ASCE[2]**

**Abstract:** Due to the presence of a relatively thick layer of miscellaneous and rubble fill overlying low strength alluvial clay, augered cast-in-place (ACIP) piles were recommended for support of the Diamond Jo Casino and City of Dubuque parking ramp. During bidding, a deep foundation contractor proposed a significant cost savings if a reduction in the pile diameters would be considered. A test pile, with the smaller diameter proposed by the contractor, was installed at the Casino site and instrumented with strain gage sister bars to evaluate the side resistance support in the fill and underlying native soils. A similar pile load test program was later performed for the Ramp project. The pile load test results provided side resistance values greater than those used in the static design and allowed the use of the smaller diameter piles proposed by the contractor.

## INTRODUCTION

In the late 1600's, the City of Dubuque Iowa became the first permanent European settlement west of the Mississippi River. The Diamond Jo Casino (Casino) and City of Dubuque parking ramp (Ramp) sites are located in previous low-lying areas along the banks of the Mississippi River known as The Port of Dubuque (The Port). Over the years, the low-lying areas were often filled with byproducts of adjacent industries. Fill materials commonly encountered beneath The Port include concrete, leather, metal, foundry sand, cinders, saw dust, wood and other miscellaneous materials.

In the Port, low strength alluvial clay is generally encountered beneath the fill underlain by loose to medium dense sand. Due to the combination of the fill and underlying low strength clay, deep foundations are often used for support of new structures. Since bedrock is about 60 meters (200 feet) below grade, deep foundations are primarily supported through side resistance in the native soils. Driven steel pipe piles, ACIP piles, pressure grouted ACIP piles and Stone Columns have been used for prior projects in The Port.

---

[1]Senior Principal, Terracon Consultants, Inc. 870 40th Avenue, Bettendorf, Iowa; PH (563) 355-0702, FAX (563) 355-4789; e-mail: wkbeck@terracon.con

[2]Senior Principal; Terracon Consultants, Inc., 135 Ambassador Drive, Naperville, IL 60540; PH (630) 717-4263; FAX (630) 357.9489; e-mail: pjharrison@terracon.com

## PROJECT INFORMATION

The Diamond Jo Casino is a two-story (no lower level), steel-frame structure with a plan area of about 6,970 square meters (75,000 square feet). Maximum column and wall loads are on the order of 2,400 kN (540 kips) and 52 kN per linear meter (12 kips per linear foot), respectively.

The post-tensioned concrete Ramp has four stories above grade and shares three common property lines with the Casino (Fig. 1). Maximum column loads are in the range of 4450 to 6670 kN (1,000 to 1,500 kips).

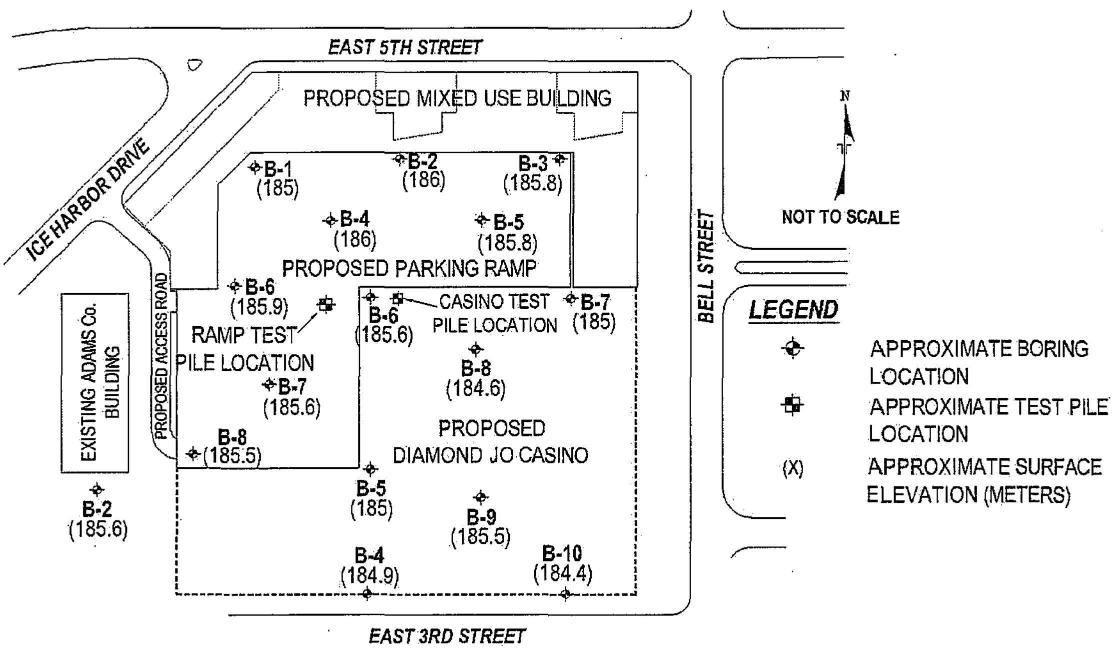


**Fig. 1. Site Plan**

The borings for the Casino site were performed in November 2006. Miscellaneous fill was present at the boring locations to depths ranging from about 3.6 to 6.1 meters (12 to 20 feet). A soft to stiff consistency alluvial clay layer, varying in thickness from approximately 0 to 6.1 meters (0 to 20 feet), was encountered beneath the fill. Primarily loose to medium dense sand was present beneath the alluvial clay to the termination depth of the borings ranging from about 16.8 to 29 meters (55 to 95 feet).

The field exploration for the Ramp was performed in March 2007. The subsurface conditions observed at the Ramp's boring locations were similar to the Casino site with miscellaneous fill to depths ranging from about 4.6 to 8.2 meters (15 to 27 feet). However, the alluvial clay layer was only present at Borings 1, 6, 7 and 8 located in the west and south ends of the site (Fig. 1). The clay layer at these boring locations extended to depths ranging from approximately 11.3 to 14.3 meters (37 to 47 feet) below grade. Loose to medium dense sand was also present beneath the fill and alluvial clay to the boring termination depths ranging from about 16.8 to 29 meters

(55 to 95 feet). No obvious rubble fill was observed at either the Casino or Ramp boring locations.

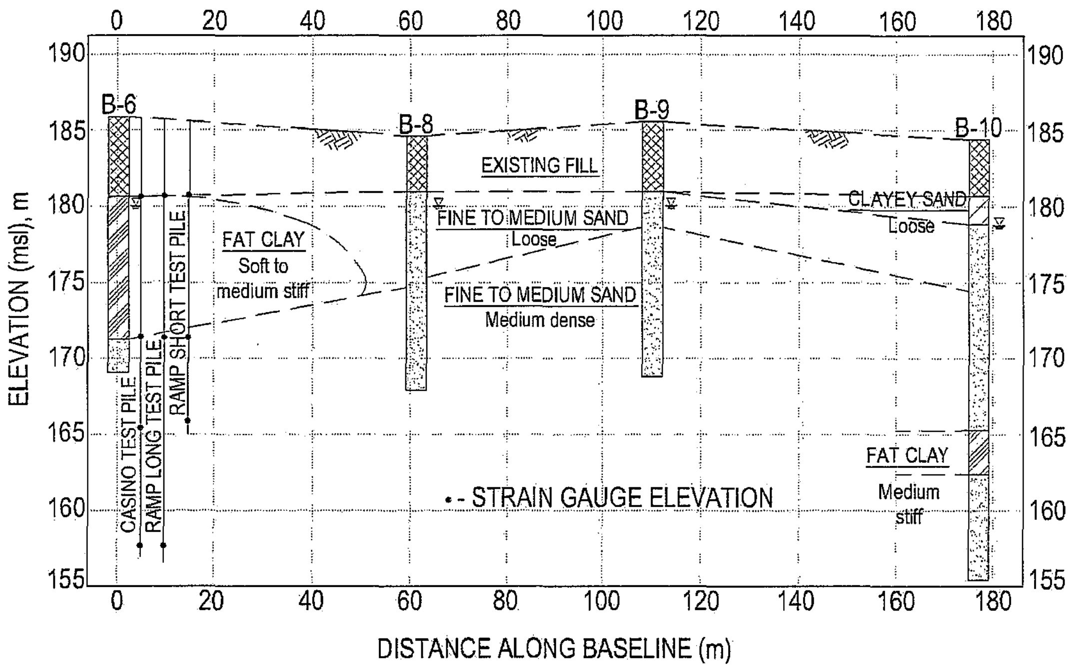


**Fig. 2. Subsurface Cross-Section – Casino Site**

## STATIC PILE DESIGN

ACIP piles have been successfully used for support of previous buildings in The Port. For this reason, ACIP piles were recommended for both the Casino and Ramp projects. Due to the variable composition and thickness of the fill, pile support within the fill was ignored. The allowable soil design parameters provided in Table 1 were used for static pile design and are based on local experience and the methods presented by Reese and Wright (1977) for drilled shafts. The allowable side resistance and end bearing values include factors of safety of 2 and 3, respectively.

The structural engineer for the Casino specified two (2) pile design capacities; 245 and 476 kN (27.5 and 53.5 tons) for a 457-mm (18-inch) diameter ACIP pile. Based on the information in Table No. 1, respective design tip elevations of about 165 and 157 meters (542 and 517 feet) corresponding to lengths of approximately 20 and 27½ meters (65 and 90 feet) were estimated for the required capacities.

For the Ramp, three (3) pile capacities for a 457-mm (18-inch) diameter ACIP pile were specified by the structural engineer; 356, 600 and 712 kN (40, 67½, 80 tons). Using the values in Table No. 1, tip elevations of about 165, 160 and 157 meters (540 feet, 525 feet and 515 feet), corresponding to pile lengths of approximately 18.9, 23.5 and 26.5 meters (62, 77 and 87 feet) were respectively estimated by the structural engineer for the required static design capacities.

**Table No. 1 Soil Design Parameters**

| Soil Description And Elevation | Allowable Side Resistance KPa (psf) | | Allowable End Bearing KPa (psf) | |
|---|---|---|---|---|
| | Casino | Ramp | Casino | Ramp |
| All Fill | --- | --- | --- | --- |
| Clay and Sand Above Elevation 167 m (560 feet) | 7 (150) | 12 (250) | --- | --- |
| Sand Between Elevation of about 165 to 167 m (540 to 560 feet) | 24 (500) | 24 (500) | 240 (5,000) | 240 (5,000) |
| Sand Between Elevations of about162 to 165 m (530 to 540 feet) | 12 (250) | 12 (250) | --- | --- |
| Below Elevation of about 162 m (530 feet) | 24 (500) | 24 (500) | 240 (5,000) | 407 (8,500) |

## CASINO PILE LOAD TEST

A deep foundation contractor bidding on the Casino project had recent experience with a nearby project where 309-mm (12-inch) diameter, 27½ meter (90 feet) long ACIP piles were used. The pile load test for that project indicated that a 309-mm (12-inch) diameter should be able to support the Casino's 476 kN (53.5 ton) pile load providing cost savings to the owner. The design team agreed to load test a 309-meter (12-inch) diameter ACIP pile to observe if the 476 kN (53.5 ton) capacity could be supported solely within the native soils beneath the fill. For this reason, the load test had to be designed to measure the side resistance/capacities within the native soils. A total test load of 1334 kN (150 tons) was targeted to account for the required load to be supported within the native soils (two times the design load) and a load of 400 kN (45 tons) that was estimated to be supported by the fill.

When compared to the other borings, it appeared that the subsurface conditions at Boring 6 (Fig. 1) would provide a conservative test pile capacity. The test and reaction piles were installed by the contractor on June 15 and 16, 2007 near the location of Casino Boring 6. The test pile was installed to a depth of about 27½ meters (tip elevation 157 meter, 517 feet). Vibrating wire strain gage sister bars were installed at elevations of about 180.5, 171, 165.5 and 158.5 meters (592, 561, 541 and 520 feet) in the test pile (Fig. 2). The load test was completed on June 29, 2007 in general accordance with the "Quick Test" procedure provided in the "Standard Test Methods for Deep Foundations under Static Axial Compression Load" (ASTM 1143). Load on the pile was measured using a calibrated load cell.

The test pile was loaded to about 1334 kN (150 tons) and about 7.1 mm (0.28 inches) of movement was observed (Fig. 3). In an attempt to reach the ultimate pile capacity, the test load was increased to the capacity of the hydraulic jack and load cell at about 1735 kN (195 tons). At the higher load, 11.4 mm (0.45 inches) of movement was measured (Fig. 3). The strain gage readings (Fig. 4) indicated that the fill was supporting about 623 kN (70 tons), or about 1/3 of the total pile load. Side resistance values obtained for the fill, alluvial clay and sand soils from the strain gage readings are provided in Table No. 2 and are substantially greater than those estimated for the static design (Table No. 1). Although the ultimate capacity of the test pile was not reached, the load test showed that the native soils could support the design load using the smaller diameter pile with a factor of safety of at least 2. Both the 245 and 476 kN (27.5 tons and 53.5 tons) piles were redesigned with 309-mm (12-inch) diameters.

## PARKING RAMP PILE LOAD TEST

The structural engineer for the ramp project included three (3) test piles in the foundation specification package, corresponding to the previously discussed static design capacities. Prior to performing the load tests, the Casino deep foundation contractor proposed a cost savings for reducing the Ramp's ACIP pile diameters. The structural engineer agreed to reduce the 457-mm (18-inch) diameter piles to a 356-mm (14-inch) diameter pile. All three test piles were instrumented with strain gage sister bars at elevations of 181.5 and 171 meters (592 and 561 feet); additional gages were installed at lower pile elevations of 166, 161.5 and 158.5 meters (545, 530 and 520 feet) in the short, intermediate and long test piles, respectively.

Based on the load limitation observed for the Casino load test, a jack and load cell capacity of about 2667 kN (300 tons) was requested for the Ramp tests; however the contractor could only provide the 1735 kN (195 tons) previously used. The long test pile was loaded on September 7, 2007 to about 1646 kN (185 tons), and movement of about 7.6 mm (0.3 inches) was observed (Fig. 3). The side resistance values from the strain gage readings for this test pile are also in Table No. 2. Over 978 kN (110 tons) was supported within the fill. Although the side resistance value for the clay was greater than used for the static design, it was almost half of the value from the Casino pile load test. Similarly, the side resistance value for the underlying sand was significantly less than observed for the Casino test pile. Variations in subsurface conditions may have contributed to a portion of these differences; however, since the fill supported so much of the test load, the test pile did not likely move far enough to develop the magnitude of side resistance observed for the Casino test pile.

The short pile was also loaded on September 7, 2007 to about 756 kN (85 tons) with observed movement of approximately 2.3 mm (0.09 inches) (Fig. 3). This test pile was subsequently loaded to about 1600 kN (180 tons) and about 5.8 mm (0.23 inches) of deflection occurred prior to abrupt movement of the pile. The maximum sustainable load, after the pile's abrupt movement was approximately 1486 kN (167 tons). Prior to abrupt movement, the strain gage readings for this test pile indicated the side resistance values shown in Table No. 2, which were lower than those observed from the Casino load test, but greater than the values used for the static design. It is possible that the abrupt pile movement was due to structural failure of the pile or from the pile breaking away from extraneous grout "tentacles" that penetrated into voids, pipes, etc. within the fill. In either case, the limited movement of the short test pile after abrupt movement likely did not fully mobilize the side resistance within the sand and clay. However, prior to abrupt movement, the short pile supported about 845 kN (145 tons) within the native clay and sand layers (Fig.

4). Therefore the smaller diameter short pile appeared suitable to support the proposed 356 kN (40 ton) design load as well as the intermediate pile load of 600 kN (67½ tons) with a factor of safety of 2. For this reason, the intermediate pile load test was not performed. By using the side resistance value for the sand obtained from the short pile load test, the long pile appeared suitable to support the 712 kN (80 ton) load. The structural engineer redesigned the piles with the smaller diameter.

**Table No. 2 Observed Side Resistance Values**

| Soil Layer | Observed Side Friction, KPa (psf) | | |
|---|---|---|---|
| | Casino Pile | Ramp Long Pile | Ramp Short Pile |
| Fill | 145 (3,000) | 163 (3,300) | 33 (685) |
| Clay | 75 (1,600) | 34 (730) | 74 (1540) |
| Sand | 90 (1,900) | 20 (410) | 31 (640) |

## CONCLUSIONS

Interpretation/evaluation of test piles extending through relatively thick layers of miscellaneous fill can be difficult. Based on our observations during construction, localized voids, buried pipes, large concrete rubble, etc. were present within the fill that were not observed at the boring locations. It is likely that the effective area/diameter of the test piles was greater within the random fill layer than the theoretical diameter. Due to the larger effective size, the fill layer supported a larger portion of the test load than expected and full mobilization of the pile within the underlying native soils could not be obtained to evaluate the ultimate side resistance and end bearing values with the jack and load cell capacity provided by the contractor. To reduce the uncertainty with the random fill layer on future load tests, the fill should be removed and replaced with loose sand at the test pile location. The use of a jack/load cell with a greater capacity would also help to reach full mobilization of future test piles.

Although test piles did not reach their ultimate capacities, the side resistance values from two of the tests were greater than those estimated for static design. Even with local experience and modifications, these pile load tests indicated that the Reese and Wright method for drilled shafts substantially underestimated the side resistance of the on-site low strength clays, and to a lesser extent, the native sands.

References

ASTM D1143-81 (1994), "Standard Test Methods for Deep Foundations under Static Axial Compression Load", American Society for Testing and Materials, Philadelphia, PA

Reese, L.C. and Wright, S.J. (1977), "Construction Procedures and Design for Axial Loading", Vol. 1, Drilled Shaft Manual, HDV-22, Implementation Package 77-21, Implementation Division, U.S. Department of Transportation, McLean, VA, 140 pp

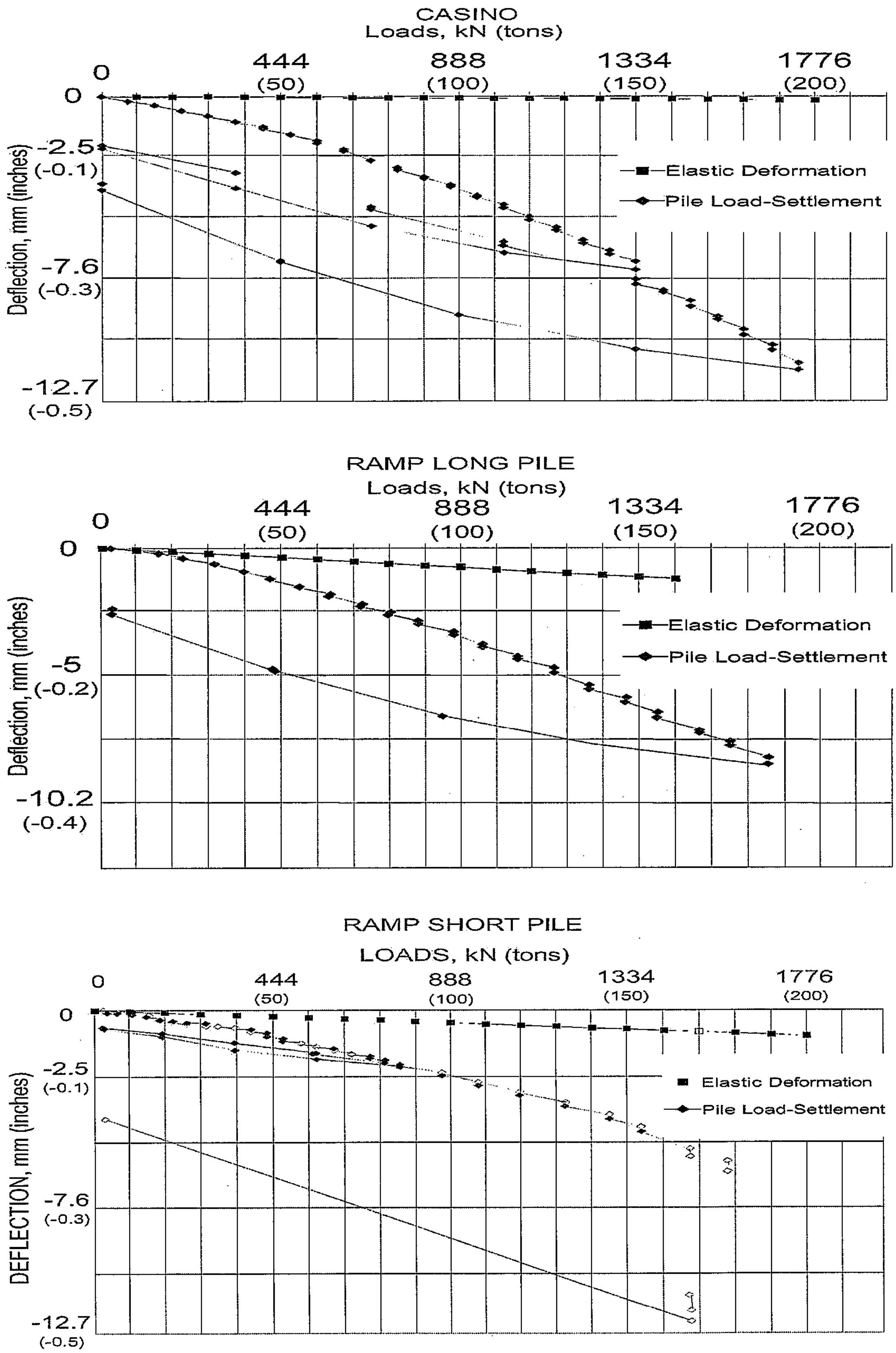


**Fig. 3. Load Deflection**

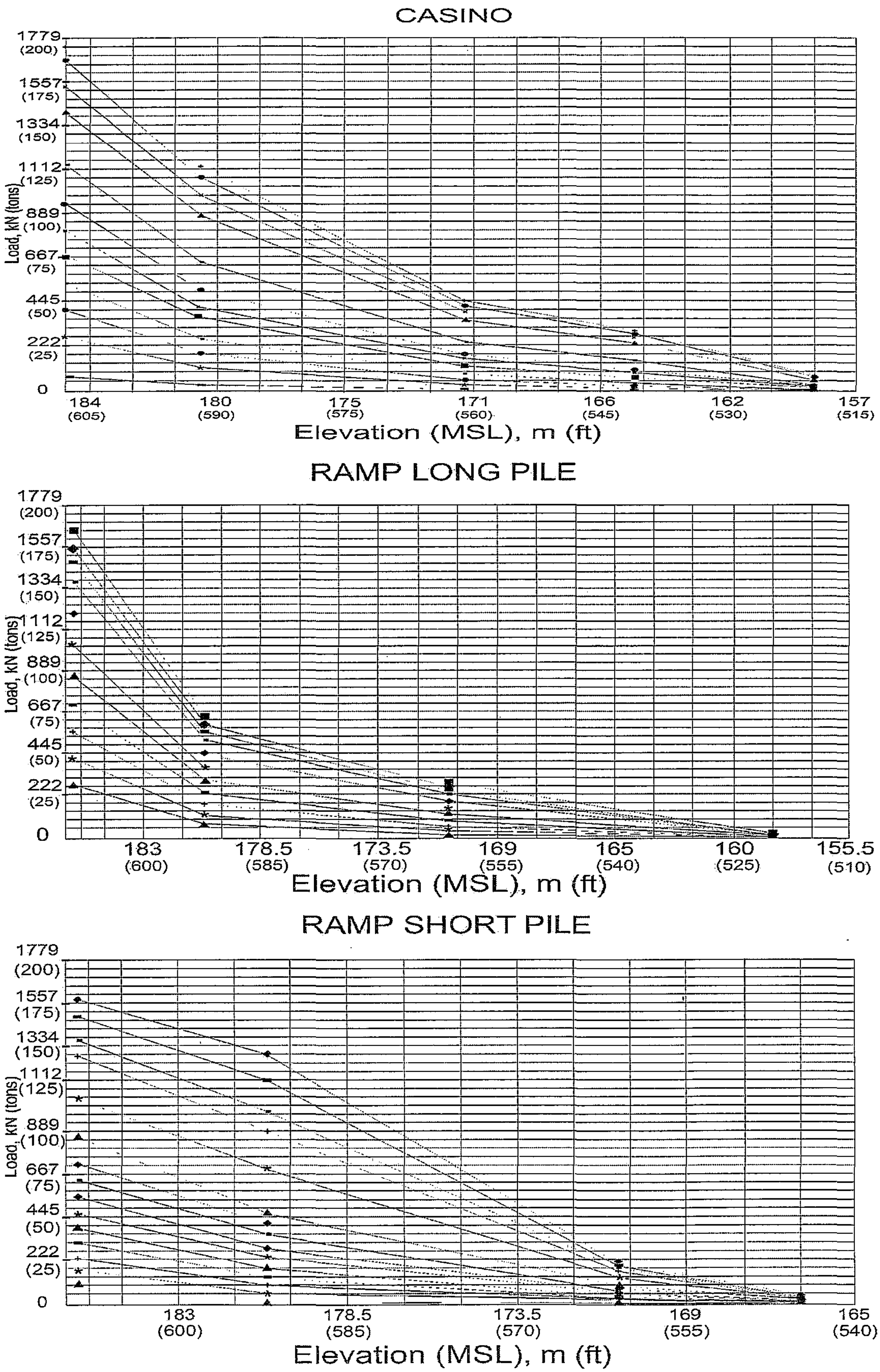


**Fig. 4. Strain Gage Readings**