The first phase of structural rehabilitation and building conservation for the Saskatchewan Legislative Building in Regina, Canada (shown in Fig. 1), began in 1998 and included the underpinning of the existing foundation. Differential movements on the order of 150 mm (6 in.) in the dome and the resulting structural distress observed in the exterior tyndal stone led to the decision to underpin the building. It was decided that new jacked piles should be installed to replace the existing 19 MPa (2700 psi) concrete "Raymond" (spirally corrugated steel shell) step-tapered piles that were installed in 1908. The new piles were designed to carry the full load of the building, with the remaining tapered piles becoming redundant structural elements. Because the original piles were too shallow to restrain building movements resulting from moisture changes in the surface clay soils, the new piles were to be driven to refusal into the underlying layer of shale.

The structural underpinning work required approximately 1800 new piles, with a design service load of 300 kN (67.4 kips), each to be installed at locations under the dome and the East, North, and South wings of the Legislative Building. The specifications also required that each pile carry a proof load of 600 kN (135 kips) for 1 hour following installation to verify that the pile was of sufficient capacity. As a result, the pile segments and connections were designed to carry the installation and proof loads of 600 kN (135 kips), which were imparted to the pile by the jacking system. The new piles were to be installed adjacent to the existing piles. Installation required excavation under the existing pile cap to allow access and sufficient room to install the new segmental piles.

An experimental program was conducted at the University of Manitoba in 1998 for the Saskatchewan Property Management Corporation (SPMC)—the government body responsible for the building. The purpose of the experimental program was to examine the behavior of the proposed segmental precast concrete piles. The piles were tested under axial, eccentric compression, and bending moment loads to determine their behavior, ultimate load-carrying capacity, and mode of failure.

**Segmental precast concrete piles**

**Pile description**

The segmental precast concrete piles consist of cylindrical segments having a length of 1 m (3 ft) and a diameter of 155 mm (6.1 in.). The segments were cast using steel-fiber-reinforced concrete (SFRC) with a 28-day nominal compressive strength of 90 MPa (13 ksi) and a steel fiber content of 40 kg/m³ (67 lb/yd³). The weight of each pile segment is 45 kg (100 lb). Each pile segment is reinforced with four 8.6 mm (0.3 in.) wires, 990 mm (39 in.) in length welded to a 20 mm (0.8 in.) diameter coil at each end as shown in Fig. 2. Recesses are provided at each end of the segments at the location of the coil as shown in the figure.

**Fig. 2—Details of segmental precast concrete piles.**
in a sufficient volume to allow the installation of a new pile. Then, a set of hydraulic jacks of 680 kN (153 kips) capacity was mounted to the underside of the pile cap as illustrated in Fig. 3. After positioning the first segment, the hydraulic jacks were extended and the segment is pushed into the ground far enough to allow installation of the following segment (also shown in Fig. 3). The connection rod is then threaded into the end of the installed segment, epoxy is placed around the bar and the top surface of the pile segment, and the next segment is threaded onto the connection rod projecting from the installed segment. The pile segments were tightened using a chain wrench to a torque of approximately 125 to 190 N-m (92 to 140 lbf-ft). This process is repeated until the pile achieves the required capacity of 600 kN (135 kips), which was typically achieved at a depth ranging from 11 to 13 m (36 to 43 ft) for the case study discussed in this article. Soil properties at the building site are given in Table 1.

Once the pile has been advanced to a sufficient depth to carry the required proof load, the pile is loaded to 600 kN (135 kips) for an hour while the settling of the pile is monitored. After completion of the proof test, each pile is locked off under a load typically equivalent to 50% of the designed service load. This preloading of the new piles is accomplished by means of sizing and installing steel transfer members while the pile is still under the applied load from the installation jacks. At this stage, the pressure on the hydraulic jacks is released, and the load is transferred to the permanent steel transfer members as shown in Fig. 3.

### Table 1—Soil structural properties at building site.

<table>
<thead>
<tr>
<th>Depth from excavated ground surface (m)</th>
<th>Type of soil</th>
<th>Undrained shear strength (kPa)</th>
<th>Horizontal subgrade modulus (kN/m²)</th>
<th>Ultimate skin friction (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 2.80 (0 to 9.2 ft)</td>
<td>Clay</td>
<td>100 (15 psi)</td>
<td>21,600 (140 kips/ft²)</td>
<td>48 (7 psi)</td>
</tr>
<tr>
<td>2.80 to 8.80 (9.2 to 28.9 ft)</td>
<td>Till</td>
<td>250 (36 psi)</td>
<td>54,000 (340 kips/ft²)</td>
<td>95 (13.8 psi)</td>
</tr>
<tr>
<td>&gt; 8.80 (&gt; 28.9 ft)</td>
<td>Shale</td>
<td>250 (36 psi)</td>
<td>54,000 (340 kips/ft²)</td>
<td>95 (13.8 psi)</td>
</tr>
</tbody>
</table>

*Excavated depth was 1.5 m (5 ft) underside of existing pile cap.

Fig. 2(a) to accommodate protective epoxy. The epoxy is required to provide corrosion protection for the threaded, high-strength steel rod that connects the individual segments and maintains continuity of the pile. This connection rod, which threads into the coils of the adjoining segments as shown in Fig. 2(b), is used to provide the continuity between the pile segments as shown in Fig. 2(c).

### Installation procedures

First, the soil underneath the pile cap was excavated in a sufficient volume to allow the installation of a new pile. Then, a set of hydraulic jacks of 680 kN (153 kips) capacity was mounted to the underside of the pile cap as illustrated in Fig. 3. After positioning the first segment, the hydraulic jacks were extended and the segment is pushed into the ground far enough to allow installation of the following segment (also shown in Fig. 3). The connection rod is then threaded into the end of the installed segment, epoxy is placed around the bar and the top surface of the pile segment, and the next segment is threaded onto the connection rod projecting from the installed segment. The pile segments were tightened using a chain wrench to a torque of approximately 125 to 190 N-m (92 to 140 lbf-ft). This process is repeated until the pile achieves the required capacity of 600 kN (135 kips), which was typically achieved at a depth ranging from 11 to 13 m (36 to 43 ft) for the case study discussed in this article. Soil properties at the building site are given in Table 1.

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### Experimental program

#### Test procedures

The experimental program consisted of four test procedures to determine the behavior of the segmental pile system under axial and eccentric loading conditions, as well as the flexural strength of the segmental system. The first test consisted of one segment, 1 m (3 ft) long, concentrically loaded to failure to determine the ultimate axial load-carrying capacity of one segment of the precast piles.

The second test also consisted of one segment, 1 m (3 ft) long, eccentrically loaded to failure to simulate the worst combined condition of axial load and moment acting on the pile. Finite element analysis (SAP 90) of a segmental pile with a length of 13.7 m (45 ft) and a possible drift of 1.6 m (5.3 ft) indicated that a load applied with an eccentricity of 16 mm (0.6 in.) created the same level of stresses that could be induced during the service life of a pile.

The third test consisted of three segments, with a
Test setup

A ±5300 kN (1.2 million lbf) capacity Material Testing Systems (MTS) closed loop testing machine was used to apply the axial load to the first specimen using a circular plate with a diameter of 360 mm and a thickness of 25.4 mm (1 in.). The load was applied using the stroke-control mode of the testing machine to trace the behavior at failure. The tested specimens were aligned to the machine through the circular plate and the threaded bar. Two holes, A and B, each with a diameter of 18 mm (0.7 in.), were drilled into the circular loading plate as shown in Fig. 4. Hole A was located at the center of the plate to apply the concentric load to the pile, while hole B was shifted by 16 mm (0.6 in.) to allow the application of the load at an eccentricity of 16 mm (0.6 in.) with respect to the center of the pile. Figure 5 shows the test setup for the 1 m specimen in the first test. The test setup for the 3 m pile subjected to concentric loading conditions is shown in Fig. 6. The fourth test consisted of two segments tested in flexure using a 270 kN (60 kips) testing machine as shown in Fig. 7. A spreader beam with two loading points located at a distance of 700 mm (28 in.) from the supports was used to apply the loads.

The instrumentation used to monitor the behavior of the tested piles under compression consisted of a combination of PI (pie-shaped) gages to measure concrete strain, and linear variable differential transducers (LVDTs) to measure lateral deflection. In the first two compression tests, four PI gages were attached around the circumference of the pile segment at midheight, as

total length of 3.0 m (9.8 ft), that were concentrically loaded to failure. The 3 m (10 ft) of unsupported length of the piles simulated support conditions that could occur in the field during installation, or over the long term as a result of desiccation of the surface soils.

The last test consisted of two segments, 2 m (6.6 ft) long, subjected to flexure using two equal concentrated loads applied at approximately the third point of the span. This was done to examine the flexural behavior of the pile-to-pile connection.
shown in Fig. 5. In the third test, six PI gages were used. Two were placed at the connection between the bottom and middle segments, and the other four were attached at mid-height of the middle pile segment as shown in Fig. 6. The deflection for the first two tests were measured at the midheight of the tested piles using two LVDTs set perpendicular to each other. For the third test, the two LVDTs were placed at the midheight of the middle pile segment. (The LVDTs were attached to a steel arm supported by the columns)

The LVDTs and PI gages were removed at a load value of 750 kN (170 kips) and 800 kN (180 kips) in the first and second test, respectively, to avoid possible damage to the instrumentation. At this stage the recorded data included only the applied load and the vertical deflection of the specimen using the machine stroke measurements.

Test results and discussion

Piles subjected to axial loads

The measured load-deflection behavior of the first three tests (which were subjected to axial loads) is shown in Fig. 8. It clearly shows the seating behavior of all the specimens at the beginning of the tests. This is evident by the large deflection values measured relative to the applied load, up to a deflection of 1.0 mm (0.04 in.). The remaining behavior of all the tested specimens was approximately linear up to failure. It is observed that the deflection values for the third test (3.0 m [9.8 ft] long specimen) were significantly higher than the corresponding values measured in the first two tests (1.0 m [3 ft] long specimen). This behavior is attributed to the increase in length of the specimen and the imperfection of the connection between the bottom and middle segments. This is obvious when a piece of paper is inserted in the joint between the two segments during loading as shown in Fig. 9. It should be noted that the deflection values for the second test were about 13% less than the corresponding values of the first test. This behavior could be attributed to the higher concrete compressive strength of the second specimen, which is confirmed by the higher measured stiffness using concrete cylinders.

All tested specimens failed by concrete splitting due to the high applied compressive stresses in the longitudinal direction parallel to the pile axis. As a result, tensile strains in the transverse directions were developed and caused failure of the concrete. The measured failure load for the first specimen was 1221 kN (275 kips). The measured failure load of the second specimen was 1678 kN (377 kips), although it was tested with an eccentric loading of 16 mm (0.6 in.). The increase in strength is due to the higher measured compressive strength of the concrete of the second specimen. The measured failure load of the third specimen was only 952 kN (214 kips). This specimen failed at the connection between the bottom and middle segments (Test 3).
Failure at the connection between the bottom and middle segments as a result of the eccentric load transfer caused by the imperfection in the connection. During the initial loading of the specimen, a gap on one side of the joint was observed. Although the gap eventually closed, the pile subsequently failed at that joint due to crushing of the concrete on the side of the joint opposite the imperfection. Figure 10 shows the failure mode for the tested specimens. Test results indicated that the axial capacity of the segmental precast piles is at least three times the design service load.

**Piles subjected to flexure**

The load-deflection behavior of the fourth specimen, tested in pure bending, is shown in Fig. 11. Prior to crack initiation, the pile segments exhibited linear behavior followed by nonlinear behavior up to failure. All the cracks were observed within the constant moment zone and began to propagate toward the supports until failure occurred. The non-linear
finite element analysis ANACAP program was used to predict the behavior of the pile segments in pure bending. The analysis accounts for the biaxial state of stresses as well as tension stiffening in the concrete. Cracking in the concrete was also accounted for by using the smeared-crack model. It is observed that the predicted deflection was higher than the measured values. This behavior is attributed to the presence of the connection between the two segments, which was not modeled in the analysis.

A typical flexural failure occurred at a load level of 22.7 kN (5.1 kips) compared with a predicted value of 24.6 kN (5.5 kips) by the nonlinear finite element model. The failure occurred at the location of the applied load through the crushing of the concrete at the top compression zone, followed by cracking of the bottom tensile zone (but not at the joints) as shown in Fig. 12.

Lateral deflection

The lateral deflection behavior of the first three tests is shown in Fig. 13. The measured lateral deflection of the second test was about 2.5 times higher than the corresponding values observed for the first test as a result of the 16 mm (0.6 in.) eccentricity used for the applied load. It can be seen that the lateral deflection of the third test increased significantly with a slight increase in the applied load up to a load value of 400 kN (90 kips). This behavior is due to the effect of the imperfection at the connection between the bottom and middle segments. After reading a load value of 400 kN (90 kips), which closed the gap between the two segments, the pile gained significant stiffness and behaved in a similar manner to the first two tests.

Conclusions

Based on the findings of this investigation, the following conclusions can be drawn:

1. The use of segmental precast concrete piles is functional, provides the required capacity with large margins of safety, and reduces the construction time in comparison with welded segmental steel pile systems;
2. The failure of the 1 m (3 ft) pile segment under axial or eccentric compression was due to the crushing of the concrete, and the failure load was more than four times the design load at service and twice the proof load;
3. The use of three unsupported segments of the precast concrete piles can reduce the ultimate load-carrying capacity by approximately 20%, due to imperfections at the connections between the pile segments;
4. The ultimate load-carrying capacity of the pile is governed by the compressive strength of the concrete; and
5. Flexural failure of the precast concrete piles did not occur at the connection between the two segments. This behavior is in excellent agreement with the predicted values given by a nonlinear finite element analysis program.

References


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