

Beam-to-Column Connections for Precast Concrete Moment-Resisting Frames
by
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Introduction

Precast concrete frame construction is not used extensively in seismic regions of the USA. The UBC [ICBO, 1991] currently permits only certain specific building systems to be used and a precast frame is not one of them. The reason is that extensive research on cast-in-place frames has led to the development of reinforcement details that provide suitable ductility, and these details are now prescribed in the UBC. In most cases, these details cannot be easily achieved in a purely precast system. The result is that most precast structures can be made to satisfy the UBC only under the guise of an "undefined structural system" which must "... be shown by technical and test data which establish the dynamic characteristics and demonstrate the lateral force resistance and energy absorption capacity to be equivalent to systems listed in Table No. 23-O for equivalent R_w values." This requirement makes approval of a precast frame very difficult. In addition, another UBC requirement calls for "reinforcement resisting earthquake-induced" forces to conform to ASTM A 706 and A 615 Grades 40 and 60 specifications which excludes prestressing steel. Since the advantages of precasting and prestressing are interlinked, this provision on prestressing inhibits the use of precast concrete.

As a result of the lack of research data, an experimental program to examine the behavior of 1/3-scale model precast concrete beam-column connections subjected to cyclic loads in the inelastic range was initiated at the National Institute of Standards and Technology in 1987. The objective of the program was to develop guidelines for the design of an economical precast moment resisting beam-to-column connection. The basic concept used for the precast connections was the utilization of prestressing steel to connect the precast elements and to provide the required shear resistance to the applied loads in the absence of corbels and shear keys.

The test program is divided into four phases. Phase I was the exploratory phase in which four monolithic specimens were tested. Two of these specimens were designed to UBC seismic Zone 2 specifications and two to Zone 4 specifications. The results from these tests serve as reference levels for the precast tests. In addition to the monolithic tests, two precast connections were tested. These specimens were designed similarly to the monolithic Zone 4 specimens. The objective of this phase was to determine the viability of the concept. Based on the results of Phase I, six precast specimens were tested in Phase II. The objective of this phase was to improve the cyclic energy dissipation characteristics of the precast specimens. Because of stiffness degradation observed in the latter stages of the tests in the earlier precast specimens, the use of partially bonded post-tensioning steel was studied in Phase III. Two precast specimens were tested in that phase.

Hybrid precast connections were studied in Phase IV. The connections are termed hybrid because they combine the use of mild or low strength steel and prestressing (PT) steel. The basic premise for this concept is that the mild steel serves as an energy dissipator while the clamping force necessary to transmit shear forces at the column face is provided by the prestressing steel. Concern was raised that the shear resistance would not be sufficient to resist the applied seismic shear loads in addition to gravity loads. To address this concern, simulated gravity loads were applied to the beams for the Phase IV tests.

Phase IV was divided into two parts, A and B. Phase IV A involved the testing of three basic configurations and six specimens. The objective of this phase was to test the concept of hybrid connections and to determine the most

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promising configuration. One of the specimens in Phase IV A which incorporated replaceable mild and PT steels was tested three times. The results were then used to determine the specimen details for Phase IV B. The primary variables in this phase were the amount and the type of mild steel. Four hybrid specimens were tested in this phase.

The remainder of this paper will concentrate on test results obtained in Phase IV B. Details from Phases I, II, III and IV A may be found in NIST reports Cheok and Lew [1990, 1991, 1993] and in Cheok, Stone, and Lew [1993].

Phase IV B: Specimen Details and Test Procedure

The design of the beam-column connections were based on a prototype 12-story moment resisting frame office structure designed for UBC seismic Zone 4. The floor plan was rectangular with dimensions of 32.92 m x 65.84 m. The story height was 3.96 m. In the design of the prototype structure, the values used in UBC equations 34-1 to 34-3 [ICBO, 1991] were:

$$V = \frac{Z I C}{R_w} W \quad (\text{UBC Eq. 34-1, [ICBO, 1991]})$$

$$C = \frac{1.25 S}{T^{2/3}} \quad (\text{UBC Eq. 34-2, [ICBO, 1991]})$$

$$T = C_t (h_n)^{3/4} \quad (\text{UBC Eq. 34-3 [ICBO, 1991]})$$

Where:

$$\begin{aligned} Z &= 4 \\ I &= 1 \\ R_w &= 12 \\ S &= 1 \\ C_t &= 0.03 \end{aligned}$$

The overall dimensions of the model beams were 203 mm (W) x 406 mm (H), and the model column dimensions were 203 mm x 203 mm. The basic connection details are shown in Figure 1. In all four specimens, the amount of PT steel was the same (3 - 13 mm diameter strands) and was located at mid-depth of the beam. The PT steel was partially bonded. In the prototype structure, the steel will be debonded through the column and for a specified distance on either side of the column and bonded for the remaining distance in the beam (bonded through mid-span of the beams). This arrangement delays or prevents yielding of the steel and provides back-up anchorage in the case of anchorage failure thereby reducing the risk of progressive collapse. Prevention of yielding of the PT steel is necessary for maintenance of the clamping force.

The beams have troughs cut out of the top and bottom which allow for field placement of mild steel bars into the ducts located in the beams and columns. The mild steel in two of the specimens consisted of Grade 60 reinforcing bars and bars made of 304 stainless steel in the remaining two. Type 304 stainless steel was selected because of its high elongation capacity at fracture, 50%. It also had similar yield strength to a Grade 60 reinforcing bar. The mild steel bars were fully bonded except for 25 mm on either side of the beam-to-column interface. These short debonded lengths in the mild steel bars and the use of 304 stainless steel bars were attempts to delay the fracture of the mild

steel bars. This was because it was felt that higher drift levels and greater energy dissipation were possible in the previous NIST tests if fracture of the mild steel was delayed.

Angles were located at the corners of the beams, top and bottom, at the beam-to-column interface. It was observed in the Phase IV A tests that the presence of these angles helped to prevent spalling of the beam corners at higher drift levels.

In addition to the loads applied to the beams, an axial load simulating the gravity load was applied to the columns. Both the beam and column loads were held constant throughout the tests. The specimens were then subjected to a loading history that was recommended for use by PRESSS [Priestley, 1992] in their test program. This loading history is drift based with three cycles at a prescribed drift level followed by an elastic cycle.

Test Results

In general, the Phase IV B specimens showed significant promise as a practical joint detail for seismic regions. They withstood cyclic loading to approximately 3% story drift before sustaining damage to the energy dissipating reinforcement (mild steel) while the PT steel remained elastic. This drift value is twice the drift envisioned by the UBC under maximum credible earthquake conditions, yet the specimens suffered essentially no damage prior to attaining this value. The hysteresis curves for the Phase IV B specimens are shown in Figures 2 - 5.

Deformed 304 stainless steel reinforcing bars with the same net area as standard Grade 60 reinforcing bars were not available at the time of the tests reported herein. Stainless round bars were thus machined with threaded lugs having outside diameter to net diameter ratios of 1.22 and 1.43, and lug width to thread pitch ratios of 0.4 and 0.19, respectively. Bars having the former ratios were used in specimen N-P-Z4 and exhibited bond failure between the stainless steel bar and the grout at a drift level of approximately 2%. In the other three specimens, failure resulted from the fracture of the mild steel bars. For the second specimen containing stainless steel bars, P-P-Z4, no significant increase in drift capacity was observed over that obtained using Grade 60 reinforcing bars. Subsequent tension tests showed that the stainless steel threaded bars broke at a strain of 30% whereas virgin rods broke at 50% strain. This reduced strain capacity was attributed to stress concentrations in the machined screw thread that was used in place of rolled-on lug pattern. Further investigations using stainless steel dissipators are warranted.

After failure of the mild steel, the specimens continued to withstand load because the PT was still intact. In one case, the specimen was tested to 6% drift, or about 4 times the drift corresponding to the UBC maximum credible earthquake. The definition of failure used for the test program was the displacement at which the resistance was less than or equal to 80% of the maximum value. The resistance was still about 65% of the maximum value at 6% drift.

After release of the load, the specimen re-centered itself. In the specimens tested to approximately 3.5% drift, the PT had suffered essentially no loss of the initial prestress, while the residual prestress was approximately $0.1 f_{pu}$ in the specimen tested to 6% drift. The initial prestress was $0.44 f_{pu}$ in all specimens. The remaining prestress proved sufficient to prevent any vertical slip at the beam-column interface under simulated gravity load. No slip was observed in any specimen. Figure 6 shows the typical force in the PT steel for specimens tested to 3.5% - 4.0% story drift. Figure 7 shows the typical force in the PT steel for the specimen which was tested to 6% story drift. The yield stress in the PT steel was approximately equal to $0.93 f_{pu}$.

Crack widths in the Phase IV B specimens were also extremely small. The widest crack in any of the specimens was 1 mm and most were in the range of 0.1 to 0.2 mm. A crack this narrow is barely visible. All cracks closed when the load was removed. The good cracking behavior was attributed to the presence of the unbonded post-tensioning and the well-anchored transverse reinforcement system. A truss model shows that the loads are transmitted through the beam by a single strut. It suggests that, at least for carriage of shear, ties are unnecessary, although they are still required for confinement at the ends of the beams.

The two monolithic specimens that were constructed and tested for reference purposes achieved story drifts of 3.4% and 3.7%. They sustained extensive crushing, diagonal cracking of the concrete, yielding of the ties, and were beyond repair. In contrast, the hybrid specimens suffered minor crushing of concrete near the armor angles at the top and bottom of the beams and spalling of the cover concrete around the mild steel bars in the column. The energy dissipated per cycle by the hybrid specimens was greater than that dissipated by the monolithic specimens up to 1.5% story drift. Comparison beyond this point is difficult due to the very different numbers of cycles to failure for each specimen. The hybrid specimens withstood 38 to 57 cycles, whereas the monolithic specimens withstood only 8.

These comparisons suggest that the hybrid specimens performed as well as, or better than, the monolithic specimens according to measures of drift capacity, strength deterioration, stiffness degradation, residual drift, cracking, damage to the concrete, and integrity of the transverse steel. In energy dissipation, the comparison is unclear. However, the hybrid specimens outperformed the monolithic ones up to a drift value implied by the UBC for the maximum credible earthquake and after that, they still continued to dissipate energy. It is worth noting that recent research [e.g. Priestley and Tao, 1993] suggests that, while some energy dissipation is important, the marginal benefits at higher drift levels are questionable, because the displacements of the structure are influenced more strongly by the individual earthquake characteristics than by the quantity of energy dissipated and by the change in the period of the structure due to the loss of stiffness.

Summary and Conclusions

A total of 22 1/3-scale model precast beam-column connections were tested. Four of these specimens were monolithic connections, the results of which served as a basis of comparison. The connections were subjected to reversed cyclic inelastic loadings. Post-tensioning was used to connect the precast elements and mild steel was used as energy dissipators. The variables included type of PT steel, location of PT steel, bonding of the PT steel, use of PT steel alone, combined use of PT and mild steels, and the amount of mild steel.

The hybrid connection has been shown to be a viable candidate for a moment resisting precast frame. It provides a way of connecting the precast members for transferring the large forces needed in severe seismic zones and takes advantage of the best features of prestressed construction and combines them with the energy dissipation of a conventional reinforced concrete structure. The use of the two separate steels, mild and PT, is essential: the PT steel, kept in the elastic range, provides the clamping force for shear resistance and the mild steel dissipates energy by yielding.

The present code regulations which prevent the use of high strength steel in seismic systems and require a precast concrete system to emulate a monolithic one are inappropriate. While emulation is clearly one option, it ignores the beneficial characteristics of precast concrete construction.

Both elastic and inelastic dynamic analyses are currently being conducted at NIST. The work includes subjecting model buildings with varying story heights to a suite of design earthquakes. The hysteretic characteristics of the connections in the building models are characterized by 7 parameters determined from an identification process of the experimental results. The ratio of the inelastic to yield displacement is approximately equal to the R_w factor used in the UBC. Based on statistical analyses of the results of this parametric study, suitable values of R_w factors for precast connections will be determined.

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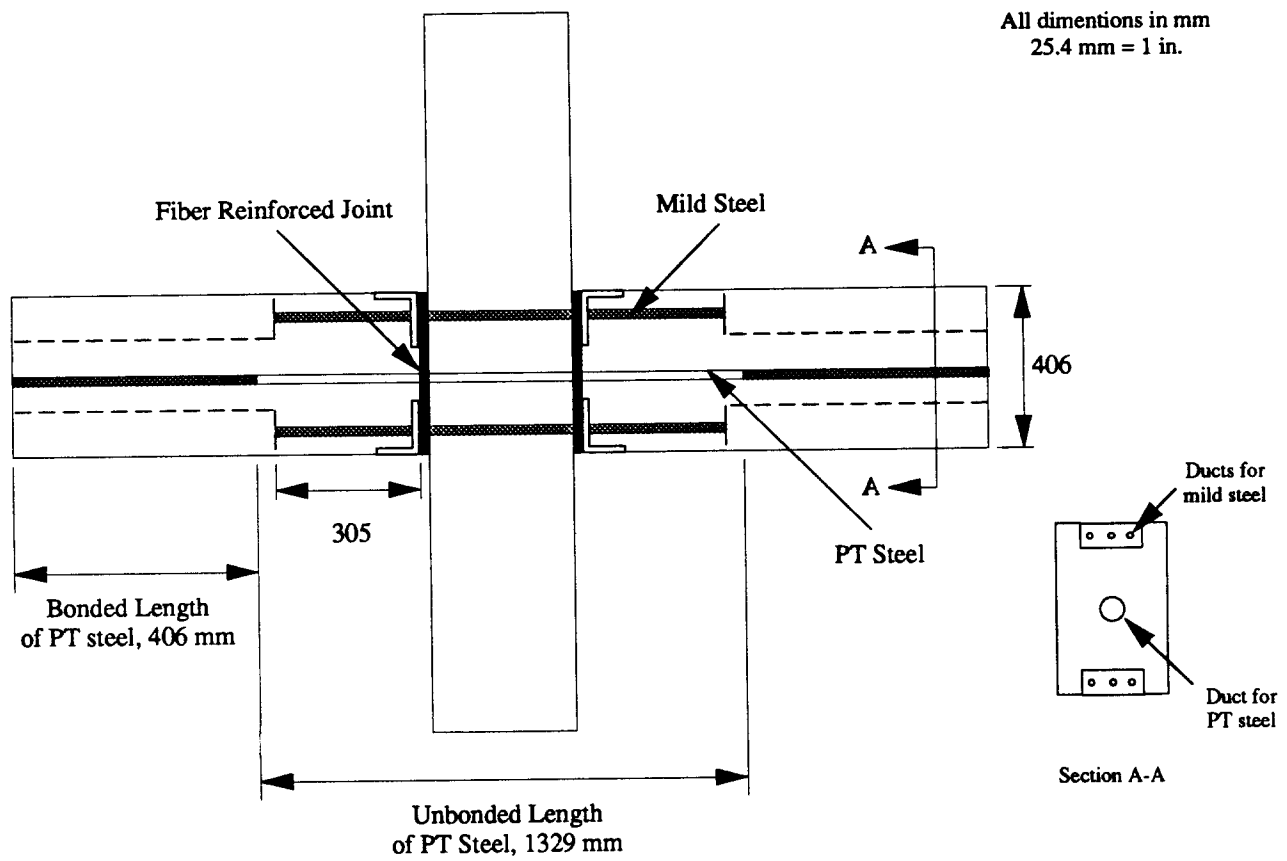


Figure 1. Basic Connection Details for Phase IV B Specimens.

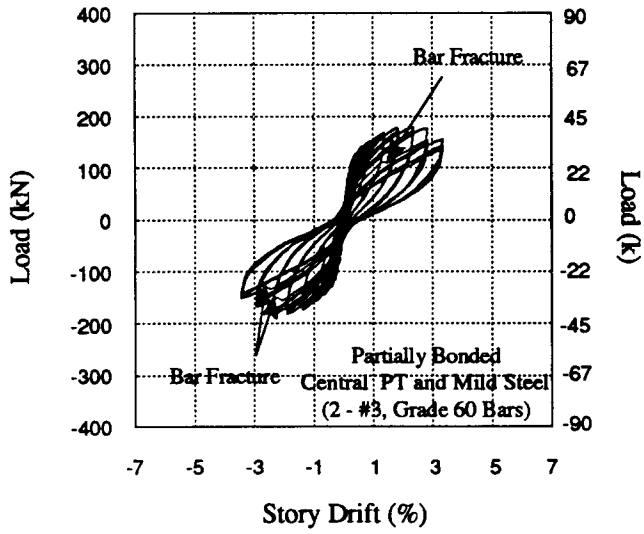


Figure 2. Hysteresis Curves for M-P-Z4.

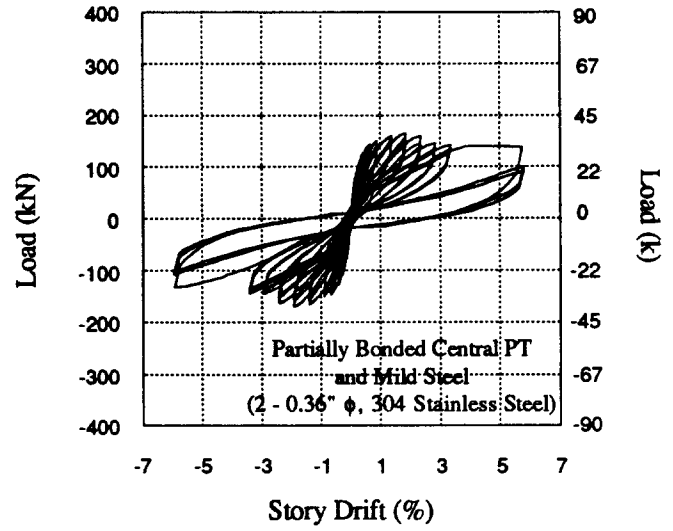


Figure 3. Hysteresis Curves for N-P-Z4.

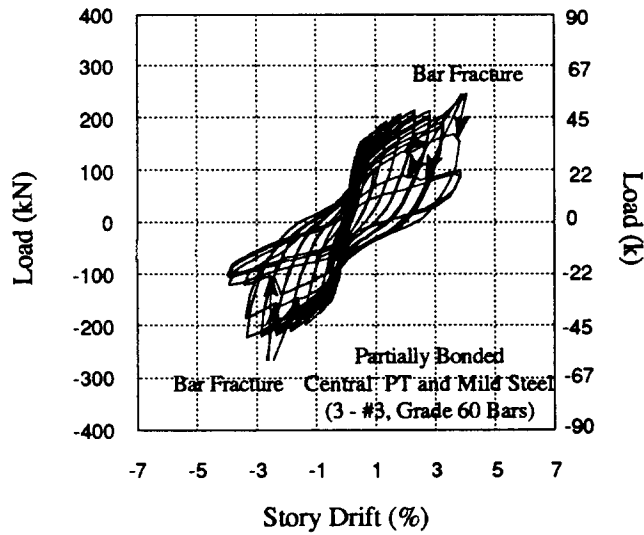


Figure 4. Hysteresis Curves for O-P-Z4.

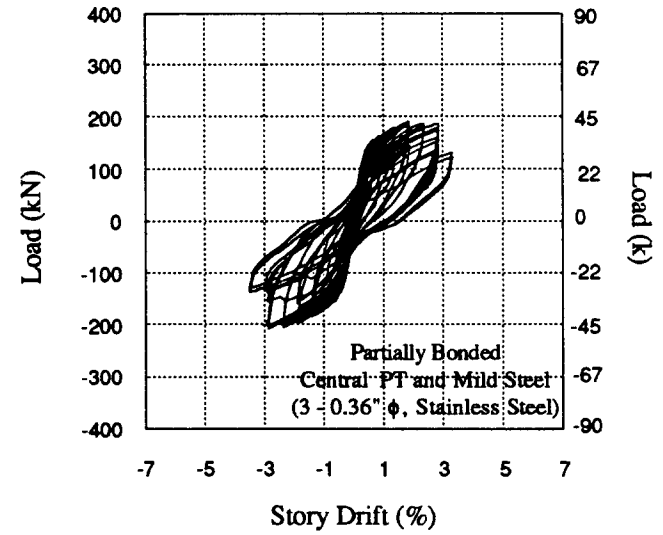


Figure 5. Hysteresis Curves for P-P-Z4.

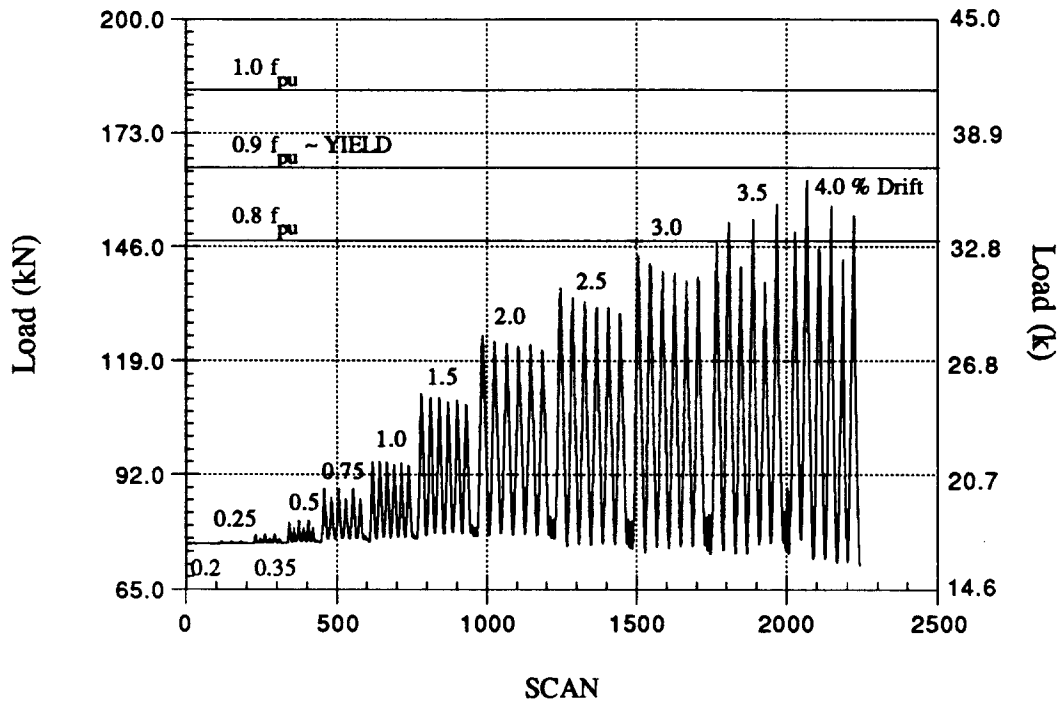


Figure 6. Typical Force in PT Steel in Specimen O-P-Z4.

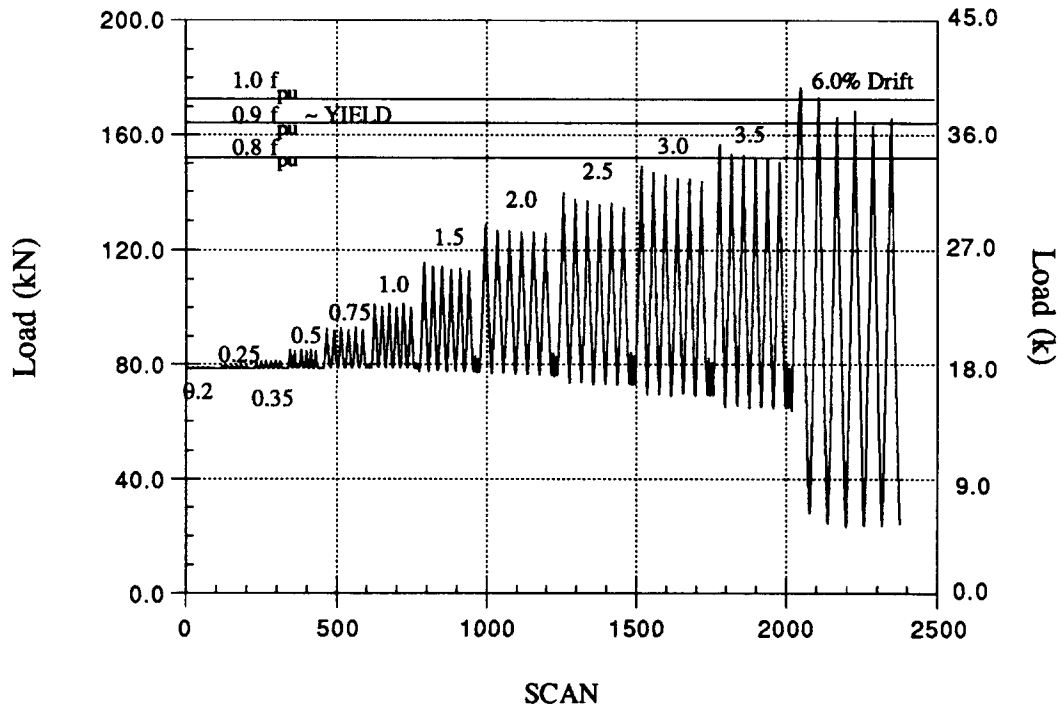


Figure 7. Typical Force in PT Steel in Specimen N-P-Z4.