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Settlement Rehabilitation of a 35 Year Old Building:

A Case Study Integrated with Analysis and Implementation

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Abstract:

This paper presents a rehabilitation project concerning the settlement of a 35 year old building. The foundation system of the northwest wing of the building consists of strip footings and slab-on-grade. Differential settlement results in significant cracking of the masonry partition walls located on the footing and hence rehabilitation of the footing is required to stabilize the foundation system. Geotechnical and structural investigations are conducted, including site borings and analytical modeling based on one-dimensional consolidation theory that is incorporated into a finite element analysis. The predictive model exhibits that the differential settlement does not cause noticeable distress for the primary structural members, whereas the continued settlement affects use of the building. Site implementation is performed with the push-pile method to terminate the continuous settlement of the foundation.

CE Database Subheadings: case studies, foundations, rehabilitation, settlement

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INTRODUCTION

Settlement of soils supporting a structural system may occur when foundation is not adequate to bear applied loads or when soil properties change due to environmental effects such as freezing-and-thawing and the level of water tables. Differential settlement is particularly harmful to constructed facilities because such soil movement can cause cracking or over-stress of structural members. Although soil-structure interaction is critical when soil properties change, it is usually ignored and ideal rigid boundary conditions are assumed when a structural design is conducted because of complexity (Mezaini 2006). Various methods have been used to improve the stability of soils when settlement is concerned, including the addition of new footings, reinforcement of soils using chemical grouts, and replacement of existing soils (Das 2007). Care should be exercised when soil-improvement methods are conducted, provided that rehabilitation of soils may induce deleterious side effects such as additional disturbance of soils. Several factors should be taken into consideration when a new foundation is added, for example, the capacity and stiffness of existing foundations and the interaction between the new and the old foundations (Poulos 2005).

D'Orazio and Duncan (1987) examined the effect of differential settlement on the behavior of steel tanks. They reported that elastic theory could be reasonably used to predict the settlement of steel tanks constructed on compressible foundation soils. Berardi and Lancellotta (2002) conducted an experimental work to evaluate the performance of oil tank structures subjected to differential settlement of soils. The local yielding of soils was accounted for, which could influence pore pressure responses of the soils. Displacement characteristics of the structures noticeably changed when supporting soils yielded. The fluctuation of water tables affected the soil behavior. Mezaini (2006) studied the contribution of differential settlement to the behavior of reinforced concrete water tanks, based on a numerical analysis. Analytical parameters considered were the geometry of water tanks, load cases, and soil properties. The predicted results were compared with available design charts. Differential settlement of existing soils caused significant changes in structural responses of the tanks. The stiffness of underneath soils governed the amount of settlement. The importance of soil-structure interaction from design perspectives. Karem et al. (2007) reported a case study related to the settlement of mine spoil materials induced by hydrocompression (e.g., rain water and drainage of surface water). A site observation was conducted to examine the propagation of cracks in a manufacturing building that was dependent upon the level of settlement. Prehydrocompression was recommended to avoid potential settlement of structures when water-induced settlement was a critical consideration.

Adequate prediction and observations of differential settlement of existing soils subjected to structural loads may provide a good understanding of the long-term performance of structural systems and may determine the need for rehabilitation, if necessary. Relatively limited information is, however, available for the analysis and modeling of building settlement. This paper presents a case study concerning the effect of differential settlement on an existing building and a rehabilitation method to preclude continuous settlement of the soils. The analysis part consists of one-dimensional consolidation theory incorporated into a finite element analysis (FEA). The proposed methodology may help practicing engineers better understand the settlement mechanism of constructed buildings and corresponding structural behavior. The site

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implementation is based on a push-pile method to provide additional foundations to the existing building.

BACKGROUND

The Family Life Center

The Family Life Center (FLC), located at North Dakota State University, Fargo, ND was completed and opened for classes in 1975. The Center is a four-story steel frame concrete and masonry building, and connects two other buildings on campus, as shown in Fig. 1(a). The area of the FLC building is 5,800 m^2 [62,430 ft^2]. The foundation of the building consists of spread footings, continuous wall foundations, and square isolated footings. All of the foundations are located below the frost-critical depth, namely, 1.5 m to 2.0 m [5 ft to 6.5 ft] below existing grade (Coulter and Heley 1973). The northwest wing of the FLC building [Fig. 1(b)] included wall foundations, footing to support masonry units, and four-inch slab-on-grade. Details of the foundations are shown in Fig. 1(c). It should be noted that the concrete for the slab was cast without form-work, which could be a common practice in the 1960's and 1970's (Cadwalader and Poland 1988).

Soil Conditions

Field testing and soil investigation were conducted in 1973 before the FLC building was built (Coulter and Heley 1973). Standard Penetration Tests (SPT) were performed at different depths in four soil borings, which were extended up to a depth of 7.5 m [25 ft] below ground surface. Several soil samples were collected in accordance with ASTM: D1586-67 and laboratory experiments were performed on selected soil samples. According to the borehole data, the soil

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beneath the foundation primarily consisted of high plasticity (CH) Fat Clay. In general, the soils encountered in shallow depths in the Fargo region are primarily alluvial soils of glacial Lake Agassiz, which once occupied the region. The average SPT – N value of the clay beneath the foundation at shallow depths was 7 (from 1.5 m to 6.0 m [5ft to 20 ft] depth below ground surface). Table 1 presents the properties of Fat clay based on field and laboratory experiments. Based on the laboratory experimental results, the undrained shear strength of the clay was 60 kPa [9 psi], saturated unit weight was 18 kN/m^3 [115 lb/ft^3] (with a water content of 33%), dry unit weight was 13.5 kN/m^3 [86 lb/ft^3], and the liquid limit and the plastic limit of the clay were 88% and 34%, respectively. The ground water table was observed at a depth of about 2.0 m [6.5 ft] even though yearly and seasonal fluctuations were observed in water tables in the Fargo region. After examining the soil conditions and laboratory experimental results, shallow continuous strip footing foundations (0.9 m [3 ft] width x 1.5 m [5 ft] depth of embedment) for the external walls of the northwest wing of the FLC building were recommended (Coulter and Heley 1973).

Settlement problem

The FLC building showed signs of severe distress as years passed and experienced significant movement in the northwest wing of the building. Temporary repair works on the building, including interior pilasters installation for roof support and other patching techniques, were conducted in 2002. However, the settlement of the exterior wall foundation and the cracking of masonry partitions in the lecture halls [Fig. 1(d)] continued even after the repair works. The performance of the building was examined in 2006 and found that the source of the structural movement was foundation settlement that caused cracking, gaps, and outward movements in the exterior walls of the building (Engelstad 2006). Laboratory experiments were conducted on Fat

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clay samples obtained from a borehole close to the northwest wing of the building (Engelstad 2006). According to the laboratory experimental results, the dry unit weight and water content of the clay about 0.3 m [1 ft] beneath the footing were 14.4 kN/m^3 [92 lb/ft^3] and 26.5% respectively (as opposed to the properties found in 1973, namely, 13.5 kN/m^3 [86 lb/ft^3] and 33%, respectively). This observation indicates that considerable amounts of compression of the Fat clay layer beneath the footings occurred between 1973 and 2006 (corresponding reduction in a void ratio was 0.17). Given the significant change in the void ratio after the building was built and the time taken (about 30 years) for the settlement to occur, it is assumed that the Fat clay was normally consolidated (NC) (virgin clay) in 1973. The amount of consolidation settlement depends on the initial in-situ stress, externally applied stress, thickness of the clay layer, and the initial void ratio and the compression index of the clay (Das 2007). Although the majority of the consolidation settlement occurred within the first few years after construction, the total consolidation settlement time was dependent upon the coefficient of consolidation of the clay and drainage boundary conditions. The primary cause of the foundation settlement with over a long period of time was the consolidation of the virgin Fat Clay (to be discussed). It should be noted that this assumption (NC clay) is justified in the settlement calculation section.

Push-pile Method

The push-pile method may be used for remediation of ground settlement problems for large buildings. The advantages of such a method include prompt execution on site, controlled noise pollution, and low costs. This method is particularly effective to terminate the differential settlement of a building and restore its elevation (previous projects done by the contractor showed durable and reliable performance). The pile system consists of steel casing piers and

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load-transfer brackets, as shown in Fig. 2. Steel casings are driven into the soil using a hydraulic jack and adjustable brackets are installed to the base of the existing foundation wall (details are available in the site implementation section). The hydraulic jacking provides stable soil movement during installation of the casing and thus disturbance of the soils can be minimized. The steel casing must reach a rigid stratum below the footing so that force transfer from the building to the foundation is achieved even though friction resistance is available (Fig. 2). The hydraulic jack attached to the steel casing can raise the foundation back to the initial elevation. Once the building is restored to the required elevation, the steel casings are bolted to the wall bracket to transfer the loads from the building to the steel piles. The casing may be filled with grouts. This method is recommended for the rehabilitation of building foundations when moderate structural loads and deep load bearing stratums are present.

ANALYSIS OF THE SETTLEMENT

This section describes the analysis of total consolidation settlement of the clay layer beneath the foundation and the rate of consolidation settlement with respect to time (starting from 1975). Table 2 presents the soil properties estimated from theoretical and empirical correlations using the available laboratory experimental results.

Consolidation Settlement

To calculate consolidation settlement, the initial void ratio (e_o) and the compression index (C_c) of the clay were estimated using known soil properties. The initial void ratio was estimated using Eq. 1, assuming that the clay was fully saturated:

$$e_o = w \cdot G_s \tag{1}$$

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where w is the water content of the clay (0.33) and G_s is the specific gravity of the soil, which was taken as 2.7. Two empirical relationships were used to estimate the compression index of clay, based on Eq. 2 (Azzouz et al. 1976) and Eq. 3 (Nishida 1956):

$$C_c = 0.4 \cdot (e_o + 0.001 \cdot w - 0.25) \tag{2}$$

$$C_c = 0.0054 \cdot (260 \cdot w - 35) \tag{3}$$

The average value of C_c obtained from both relationships was used for the settlement calculations (C_c = 0.26). Figure 3 shows the cross section of the strip footing foundation of the FLC building [Fig. 1(c)]. The vertical load from the wall at ground level (Q) varied from 85 kN/m [483 lb/in] to 100 kN/m [568 lb/in] (per meter length of the wall). The width (B) of the strip footing was 0.9 m [3 ft] and it was embedded at 1.5 m [5 ft] depth (D) from ground surface. The location of the water table fluctuated with seasonal changes, however a conservative depth of the water table (1.5 m [5 ft] from ground surface) was assumed for this study. The thickness of the Fat Clay layer (H) beneath the base of the footing was about 4.0 m [13 ft]. A stiff clay layer was found after a depth of 5.5 m [18 ft] from the ground surface and it was assumed that the stiff clay layer did not contribute to the observed consolidation settlement. The rationale for this assumption is twofold: (1) the stiffness of the stiff clay layer is higher than that of the Fat Clay layer and (2) the applied stress from the foundation decreases with depth (less external stress on stiff clay when compared to Fat Clay).

The clay layer beneath the footing was divided into 20 sub-layers and the 2:1 stress reduction method was used to calculate the incremental vertical effective stress in each layer (Das 2007), as shown in Fig. 3. In the middle of any sub-layer, a depth of z beneath the footing, the initial,

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incremental, and final vertical effective stresses (σ_{vo}', $\Delta\sigma_v$, and σ_{vf}' respectively) were calculated using Eqs. 4 to 6:

$$\sigma_{vo}' = \gamma_{sat} \cdot D + \left(\gamma_{sat} - \gamma_w\right) \cdot z \tag{4}$$

$$\Delta\sigma_v = \frac{Q}{\left(B + z\right)} \tag{5}$$

$$\sigma_{vf}' = \sigma_{vo}' + \Delta\sigma_v \tag{6}$$

where γ_{sat} and γ_w are the saturated unit weight of the clay and unit weight of the water, respectively. The settlement of each sub-layer (S_c) was calculated using the consolidation equation applicable to NC clays (Eq. 7):

$$S_c = H_o \cdot \frac{C_c}{\left(1 + e_o\right)} \cdot log\left[\frac{\sigma_{vf}'}{\sigma_{vo}'}\right] \tag{7}$$

where H_o is the thickness of the sub-layer. The total consolidation settlement was a summation of the settlement of all sub-layers. A similar procedure was used to calculate the settlement of the slab-on-grade in the northwest wing of the building, based on the dead and live load distributions on the floor of 2.4 kPa [0.35 psi] and 1.9 kPa [0.28 psi], respectively (ASCE 2005).

Rate of Consolidation Settlement

Figure 4 shows the consolidation settlement of the wall foundation of the building (Fig. 1c) as a function of time. To calculate the time-rate of consolidation settlement, the permeability (k) of the clay was estimated using an empirical relation (Dolinar 2009):

$$k = C \cdot e_o^{\,D} \tag{8a}$$

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$$C = exp\left[-5.51 - 4 \cdot ln(PI)\right] \tag{8b}$$

$$D = 7.52 \cdot exp\left[-0.25 \cdot \frac{(w\% - PL)}{PI}\right] \tag{8c}$$

where PL and PI are the plastic limit and plasticity index of the clay, respectively. The estimated value of the permeability was 2.0×10^{-10} m/s [6.6×10^{-10} ft/s]. The coefficient of volume compressibility of clay m_v was calculated using Eq. 9 for each sub-layer:

$$m_v = \frac{\left(\frac{S_c}{H_o}\right)}{\Delta\sigma_v} \tag{9}$$

The average value of m_v obtained from all sub-layers was used for the time-rate of consolidation settlement calculations ($m_v = 1.0 \times 10^{-3}$ m^2/kN [48×10^{-3} ft^2/kip]). The coefficient of consolidation of the clay (C_v) and the non-dimensional time factor (T_v) were calculated using one dimensional consolidation theory shown in Eqs. 10 and 11 (Terzaghi and Peck 1967):

$$C_v = \frac{k}{m_v \cdot \gamma_w} \tag{10}$$

$$T_v = \frac{C_v \cdot t}{{H_{dr}}^2} \tag{11}$$

where H_{dr} is the maximum drainage distance and t represents the time after the construction of the building. The stiff clay beneath the Fat Clay layer was assumed to be impermeable and the value of H_{dr} was set equal to the thickness of the Fat Clay layer (one-way drainage). The degree of consolidation (U) was correlated to the time factor (T_v) through Eq. 12 (Das 2007):

$$T_v = \frac{\pi}{4} \cdot U^2 \qquad \text{for } U < 0.6 \tag{12a}$$

$$T_v = 1.781 - 0.933 \cdot log(100 - U\%) \qquad \text{for } U\% > 60\% \tag{12b}$$

The consolidation settlement (S_t) at any time t was then calculated using Eq. 13:

$$S_t = S_{c,total} \cdot U \tag{13}$$

where $S_{c,total}$ is the total consolidation settlement of the Fat clay layer.

Calculated Settlement

As shown in Fig. 4, the total settlement of the wall footing was approximately 150 mm [6 in]. The majority of the settlement, however, occurred in clay layers immediately beneath the footings as the applied stresses (due to the building weight) decreased with depth. The calculated settlement of the first two layers (about 400 mm [16 in] thickness) immediately after footing was 34 mm [1.3 in]. The change in void ratio (Δe) of the clay layer may be calculated using Eq. 14 (Das 2007).

$$\frac{\Delta e}{1+e_o} = \frac{\Delta H}{H} \tag{14}$$

where e_o is the initial void ratio (0.891), H is the thickness of the clay layer (400 mm [16 in]), and ΔH is the settlement. Eq. 14 provided 0.16 for Δe. The change in void ratio (from 1973 to 2006) may also be calculated based on laboratory experimental results using Eq. 15 (Das 2007).

$$\Delta e = \Delta w \cdot Gs \tag{15}$$

where Δw is the change in water content (33% - 26.5% from 1973 to 2006). Eq. 15 predicted 0.17 for Δe, which was consistent with the consolidation settlement calculations. It is, therefore, concluded that the clay was normally consolidated when the building was built.

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Stiffness Variation of Soils

Figure 5 shows the variation of soil stiffness with respect to time. The net stiffness of the soil was estimated using Eq. 16:

$$K_{net} = \frac{P}{\delta_{net}} \tag{16}$$

where K_{net} is the net stiffness of the soil, P is the applied load effect, and δ_{net} is the net increment of settlement between two arbitrary time frames (obtained from the consolidation theory). The net stiffness represents the decreased settlement of the soil when the year increased [Fig. 5(a)]. The effective secant stiffness of the soil is shown in Fig. 5(b), which is related to the stiffness based on the total settlement, rather than the net settlement within two time frames. It should be noted that the effective stiffness was obtained from the settlement of the wall foundation minus that of the slab to exclude the rigid body movement of the members. A 68% decrease in the effective stiffness was observed for the first 10 years; then the stiffness was levelled off, as shown in Fig. 5(b).

Load Carrying Capacity of Piles

The skin friction resistance of the piles in saturated clays under undrained conditions (Fig. 2) may be estimated using Eq. 17 (Das 2007):

$$Q_s = \pi \cdot D_p \cdot L \cdot \alpha \cdot C_u \tag{17}$$

where Q_s is the skin friction resistance force, D_p is the outside diameter of the pile, L is the length of the pile, α is the adhesion factor between pile and clay, and C_u is the undrained shear strength of the clay. Although the adhesion factor (α) varies with depth, the relationship provided by

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Randolph and Murphy (1985) was used to calculate an average value (α = 0.65). Using a conservative undrained shear strength (60 kPa [8.7 psi] at shallow depths), the shaft resistance of a single pile was calculated to be 125 kN [28 kip]. This will provide about 100 kN/m [568 lb/in] resistance along the exterior walls. It should be noted that the values used in the calculations are on the safe side, given that the actual load varied from 85 kN/m [483 lb/in] to 100 kN/m [568 lb/in], and hence the estimate for Q_s is conservative.

NUMERICAL MODELING

To predict the structural response of the northwest wing of the FLC building, an FEA was conducted using the general-purpose FEA package ANSYS. The following summarizes the details of the modeling approach.

Preprocessing

Elements and materials

Three-dimensional elastic beam elements were used to represent the foundation of the building, including the wall footing and the footing supporting the masonry wall [Fig. 1(c)]. The element includes six degrees of freedom: three translational and three rotational degrees of freedom. The geometric properties of the foundation [Fig. 1(c)] were represented by the thickness and the moment of inertia for the beam elements. Four-node elastic shell elements were used to model the slab-on-grade. This element permits out-of-plane displacement so that the deflection of the slab-on-grade could be predicted. The shell element has the same degrees of freedom as the elastic beam element and hence there would be no discrepancy in terms of degrees of freedom between these elements. To simulate the soil effect beneath the four-inch slab [Fig. 1(d)], the

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elastic foundation stiffness was defined for the shell elements in the out-of-plane direction: the pressure required to provide a unit displacement of the foundation (ANSYS 2009). The effective soil stiffness shown in Fig. 5(b) was converted to the elastic foundation stiffness with unit area for this study. The strength of concrete was 25 MPa [3,600 psi] with an elastic modulus of 31 GPa [4,500 ksi] and a Poisson's ratio of 0.25. The effects of environment such as temperature and moisture were not considered for the model.

Geometric modeling and boundary conditions

The northwest wing of the building was modeled with a combination of the beam and shell element to represent the foundation shown in Fig. 1(c). For modeling convenience, the wall foundation was assumed to be rigidly connected to the four-inch slab, rather than detailed modeling of the connection. Such an ideal assumption was not able to simulate the local rotation of the slab that could take place on site. The foundation and the slab were assumed to be fixed at the main building structure [Fig. 1(b)] and necessary nodes were constrained in all translational degrees of freedom accordingly. The load applied to the foundation was 92 kN/m [522 lb/in] on average and the load for the slab was 4.3 kN/m^2 [0.6 psi], as was done in the settlement analysis.

Postprocessing

Settlement of foundation

Figure 4 shows the settlement of the wall footing with respect to time up to 40 years. Good agreement between the FEA and the one-dimensional consolidation theory was obtained with an average error of 3.0%. It should be noted that the settlement in Fig. 4 (FEA) was obtained at the very end of the northwest wing of the building where the maximum displacement took place. A

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rapid increase of 35.4 mm [1.4 in] and 32.6 mm [1.3 in] in the settlement was observed for the consolidation theory and FEA for the first year, respectively. The settlement, then, gradually increased up to approximately 150 mm [6 in] when the time increased to 20 years and the settlement tended to be stabilized, as shown in Fig. 4. The predictive settlement reasonably agreed with a visual inspection along the perimeter of the northwest wing of the building (varying 100 mm [4 in] to 150 mm [6 in] depending upon the locations) that was conducted prior to the rehabilitation work. Figure 6(a) shows the settlement profiles along the foundation inside the lecture hall [Section-1 in Fig. 1(d)]. A trend of differential settlement was clearly observed along the foundation that supported the masonry wall between the two lecture halls [Fig. 1(d)]. Such a differential settlement caused the serious cracking of the masonry units. The differential settlement across Section-2 [Fig. 1(d)] was not as significant as that of Section-1, as shown in Fig. 6(b). The waved-settlement across Section-2 is attributed to the difference of the sectional properties between the foundation and the slab-on-grade [Fig. 1(e)]. This observation supports the fact that there was no noticeable cracks in the floor of the lecture halls.

Influence of the settlement on structural members

The predicted bending stress of the foundation along Section-1 is shown in Fig. 7(a). The beam foundation exhibited positive and negative bending stresses because of the double curvature of the member [Fig. 6(a)]. The predicted stresses were less than 5 MPa [725 psi] that slightly exceeded the modulus of rupture of the footing concrete (3 MPa [435 psi]). It should, however, be noted that actual cracking of the footing member could not be expected on site, provided that the predicted stresses were based on a full-loading condition including the dead and live loads. The sudden decrease in bending stresses beyond 10 m [33 ft] in Fig. 7(a) is attributed to an increase in the moment of inertia of the foundation; in other words, the sectional properties changed from the foundation without wall to that with wall, as shown in Fig. 1(d),(e). The development of the peak stresses with respect to time is shown in Fig. 7(b). The stress profiles were almost symmetric in positive and negative bending even though the locations were not the same, as shown in Fig. 7(a). A rapid increase in peak stresses of the footing was observed for the first year: 65% of the stress in 40 years was noticed during the first year, as shown in Fig. 7(b). The peak stresses were leveled off after 10 years of service. It is, therefore, concluded that the differential settlement of the soils beneath the foundation could not substantially influence the behavior of the structural elements even though it affected the behavior of the masonry wall that showed significant cracking.

IMPLEMENTATION OF FOUNDATION STABILIZATION

Figure 8 shows the site implementation to stabilize the continuous settlement of the building. To install the push-piles, the site was excavated until the existing footing was exposed [Fig. 8 (a),(b)]. The push-pile system used here included a high-strength steel casing (Φ = 150 mm [6 in]) and a load transfer bracket tucked under the foundation pile cap that was designed to lift up the existing foundation during a jacking operation, as shown in Fig. 8(c). The foundation bracket was bolted to the existing footing and the pile was driven hydraulically into the soil. The steel casing, a spacing of 1.2 m [4 ft] on center, was continuously driven until a suitable load-bearing stratum was achieved (about 7.0 m [23 ft] below the foundation level) and the initial elevation of the foundation was restored. The pile was cut and the hydraulic jack was removed, as shown in Fig. 8(d). The loads of the building were then transferred to the rigid foundation through the push-pile system. The loads were partially resisted by the skin friction between the piles and clay

(Fig. 2), given that the clay had already been consolidated to the applied stress level and the base resistance from the stiff clay layer was located beneath the Fat Clay layer. The piles were grouted to fill the void [Fig. 8(e)]. Upon completion of the push-pile system, the site was recovered, as shown in Fig. 8(f). After the rehabilitation work, the structural movement of the FLC building was terminated.

CONCLUDING REMARKS

This paper has presented a case study regarding the settlement of the northwest wing of the Family Life Center situated at North Dakota State University, including theoretical analysis and corresponding implementation on site. Geotechnical investigations were conducted to examine the consolidation history of the building, based on the soil boring data and one-dimensional consolidation theory. Numerical models were developed to predict the effect of differential settlement on the behavior of structural members consisting of the foundations and slab-on-grade. The predicted settlement was stabilized after 20 years of construction, whereas continuous increases in the settlement were observed on site, resulting in significant cracking of the masonry units in the lecture halls. The discrepancy between the theory and the site may be attributed to the seasonal change of water tables that could be interacted with cold temperature or insufficient compaction of the soils. A site rehabilitation project was conducted with the push-pile method to terminate the settlement of the northwest wing of the building.

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Table 1. Soil properties (beneath the footing) based on field and laboratory experiments

Parameter	Soil property
Soil type	Fat clay
ASTM classification	CH
Thickness (m) [ft]	4 [13]
Dry unit weight (kN/m^3) [lb/ft^3]	13.5 [13]
Saturated unit weight (kN/m^3) [lb/ft^3]	18 [115]
Water content (%)	33
Liquid limit (%)	88
Plastic limit (%)	34
Undrained shear strength (kPa) [psi]	60 [8.7]
Average SPT-N value	7

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Table 2. Estimated soil properties based theoretical and empirical correlations and laboratory experimental results

Soil property	Value
Void ratio	0.89
Compression index (C_c)	0.26
Coefficient of permeability (m/s) [ft/s]	2.0×10^{-10} [6.6×10^{-10}]
Coefficient of compressibility (m^2/kN) [ft^2/kip]	1.0×10^{-3} [48×10^{-3}]
Coefficient of consolidation (m^2/s) [ft^2/s]	2.0×10^{-8} [21.5×10^{-8}]

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Table 3. Comparison between consolidation theory and FEA

	Consolidation theory	FEA
Objective	To predict the settlement of the wall footing	To predict the structural response of the building
Assumed properties	Detailed soil properties were accounted for (Table 1)	Soil properties were modeled using the net stiffness (Eq. 16)
Loading condition	Maximum load of 100 kN/m [568 lb/in] was taken along the wall footing	Average load of 92 kN/m [522 lb/in] was used to examine structural behavior
Maximum settlement	152.9 mm [6.0 in] in 40 years	156.4 mm [6.1 in] in 40 years
Limitation	The footing supporting the masonry wall was not included due to its insignificant effect	Ideal boundary conditions were applied

List of Figures

Figure 1a

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Figure 1b

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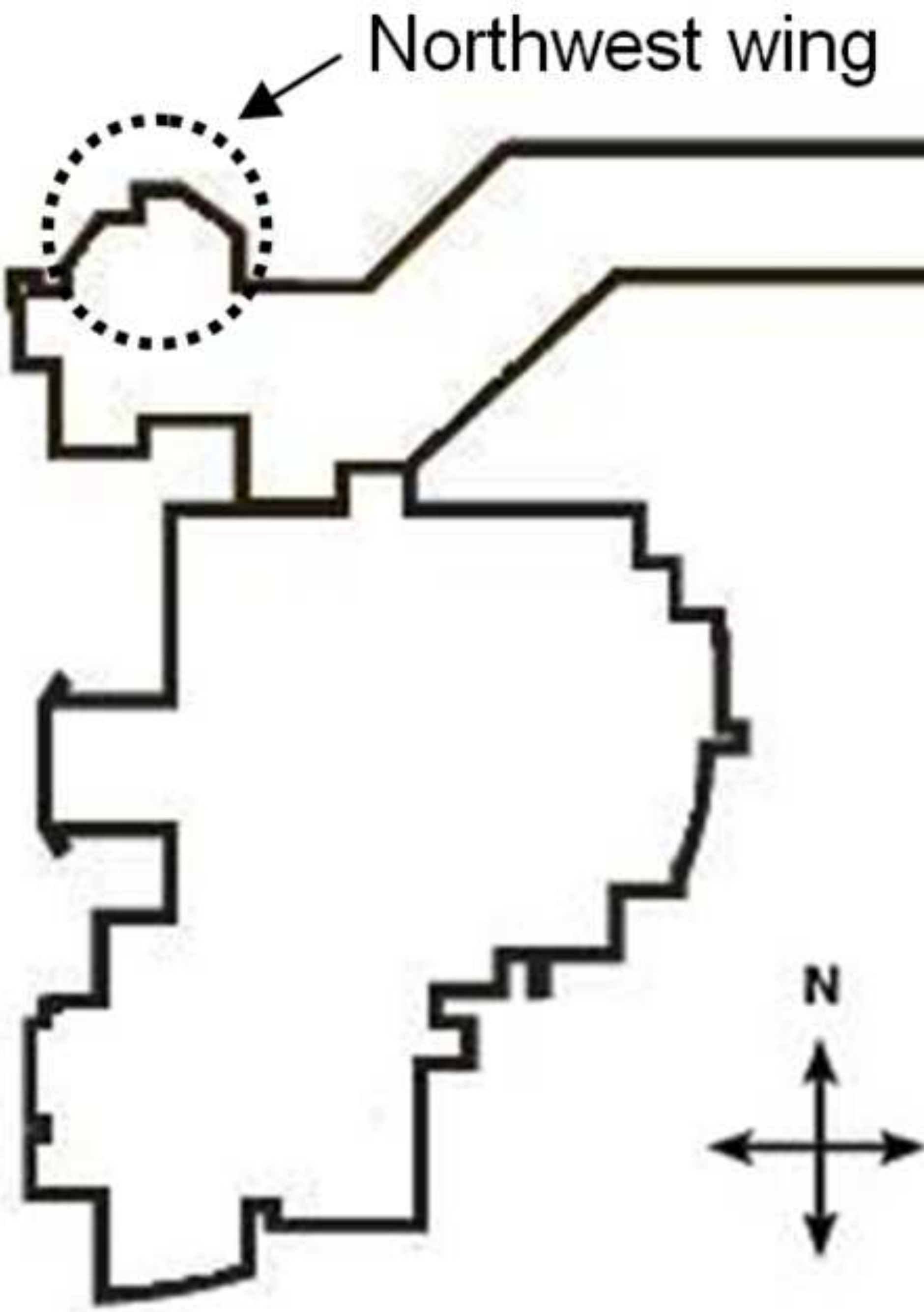

Figure 1c

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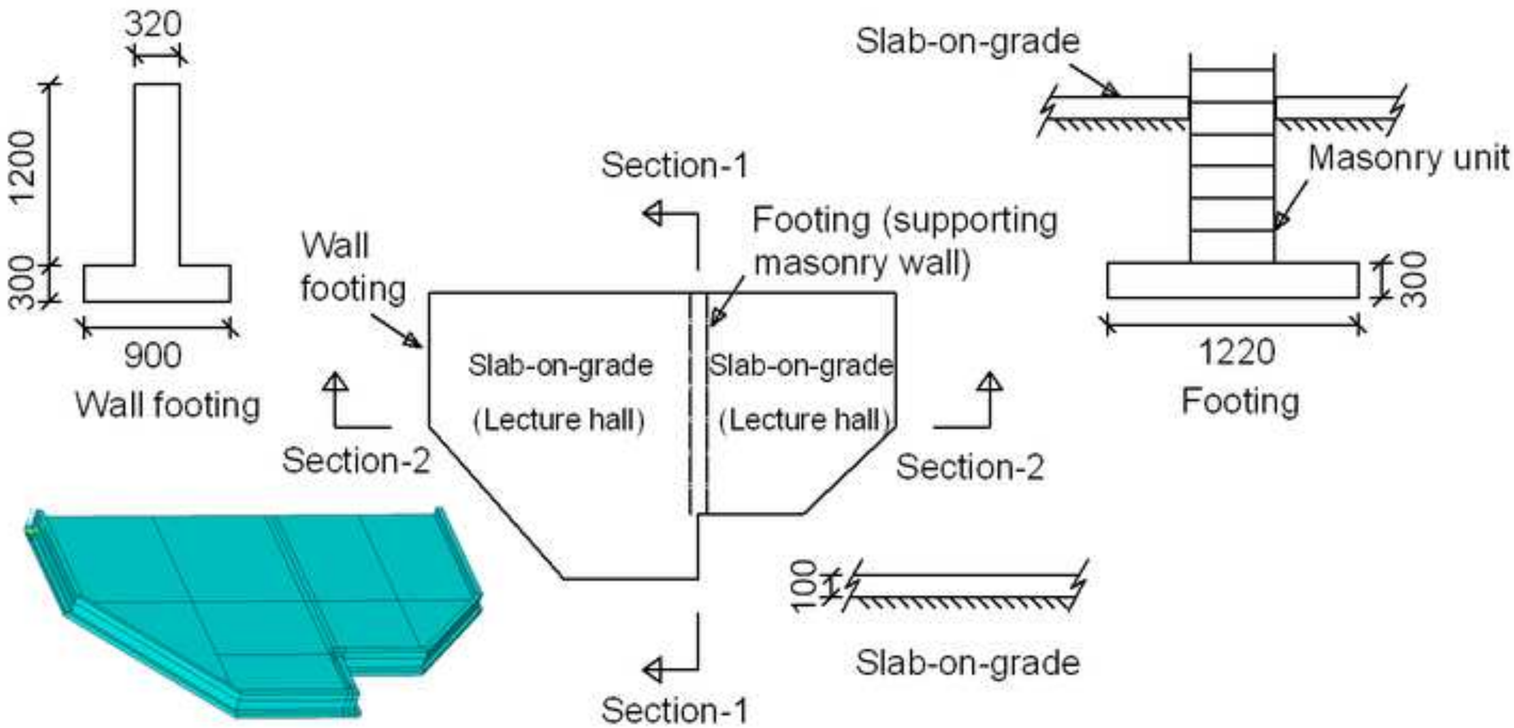

Figure 2

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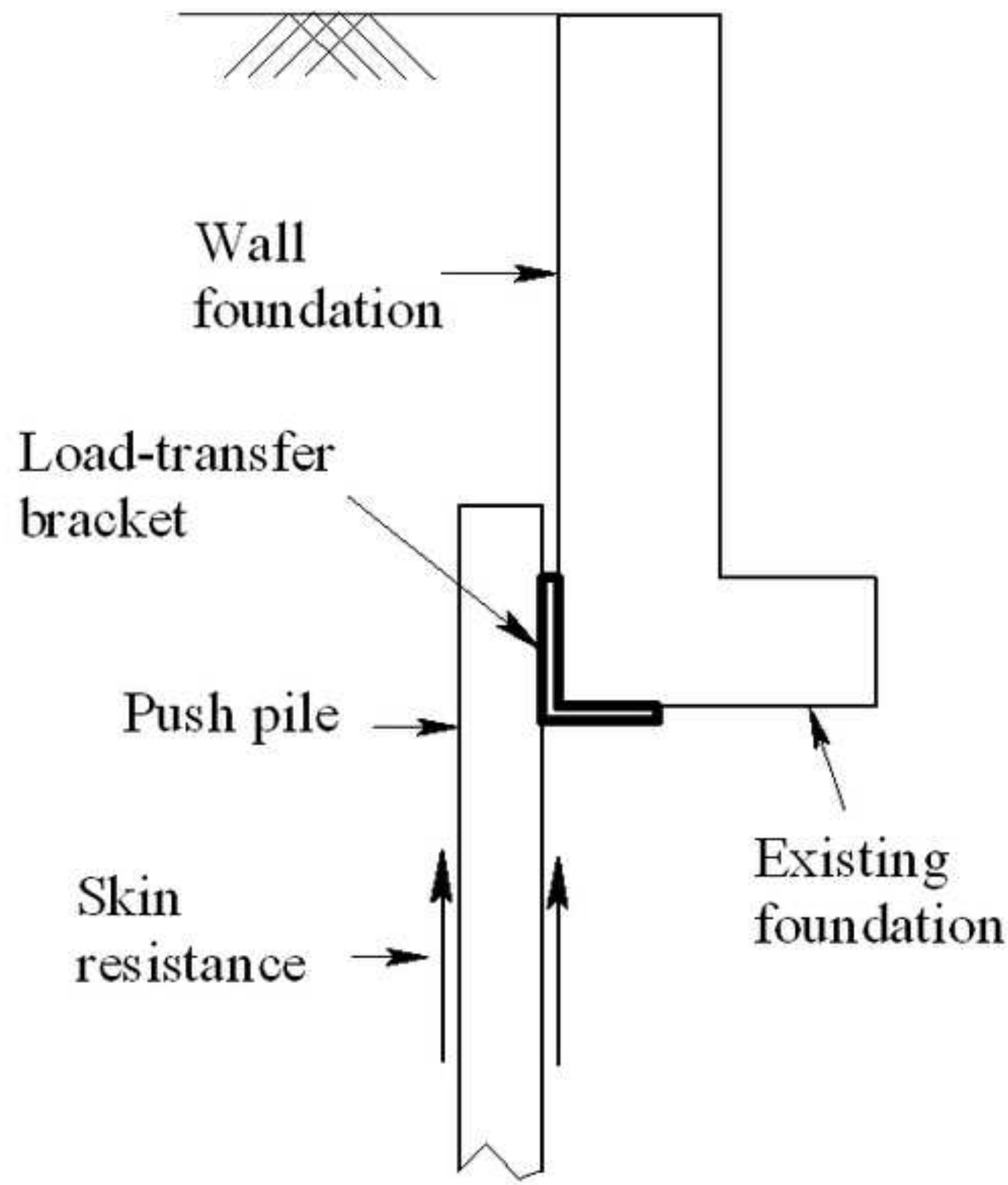

Figure 3

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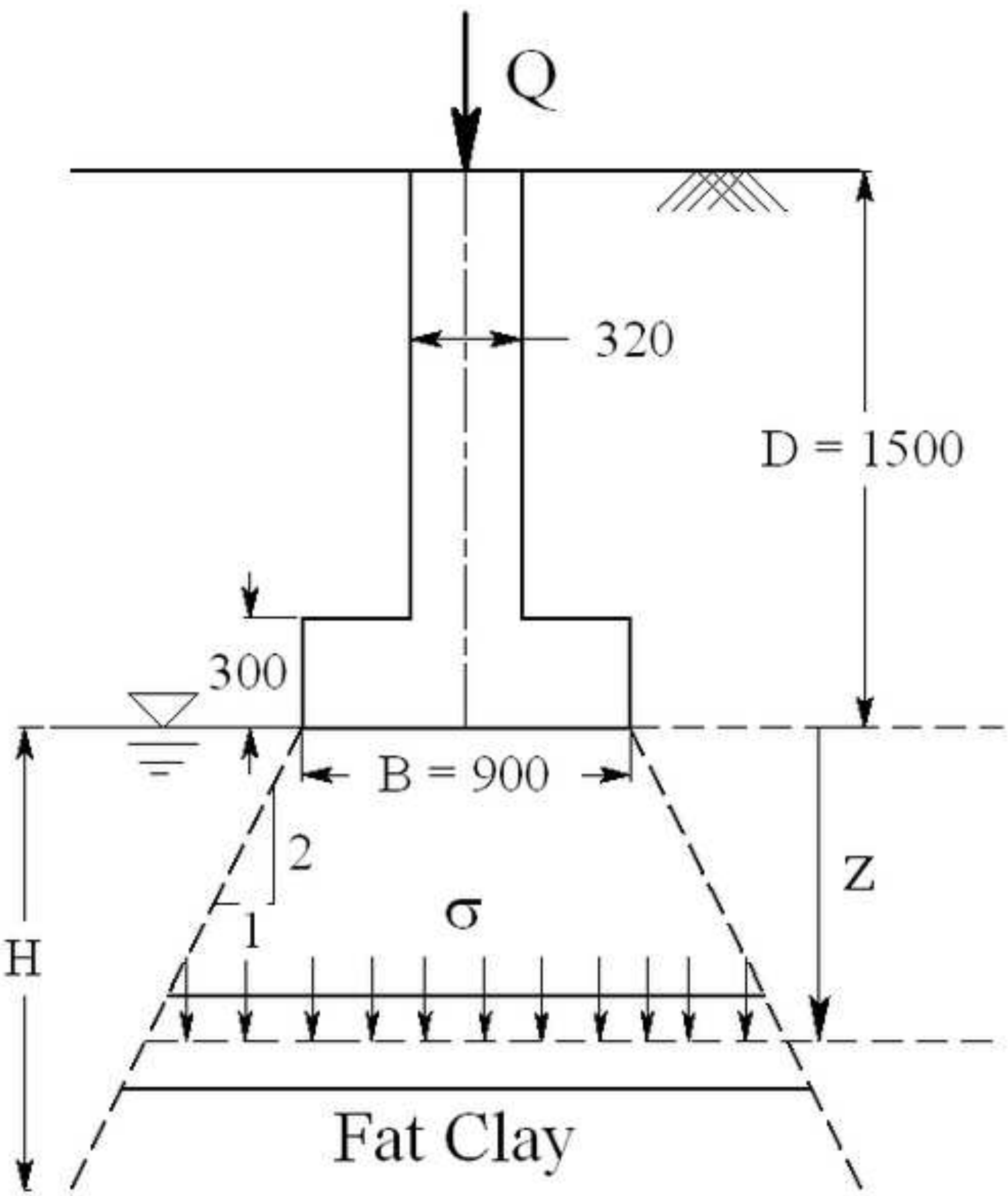

Figure 4

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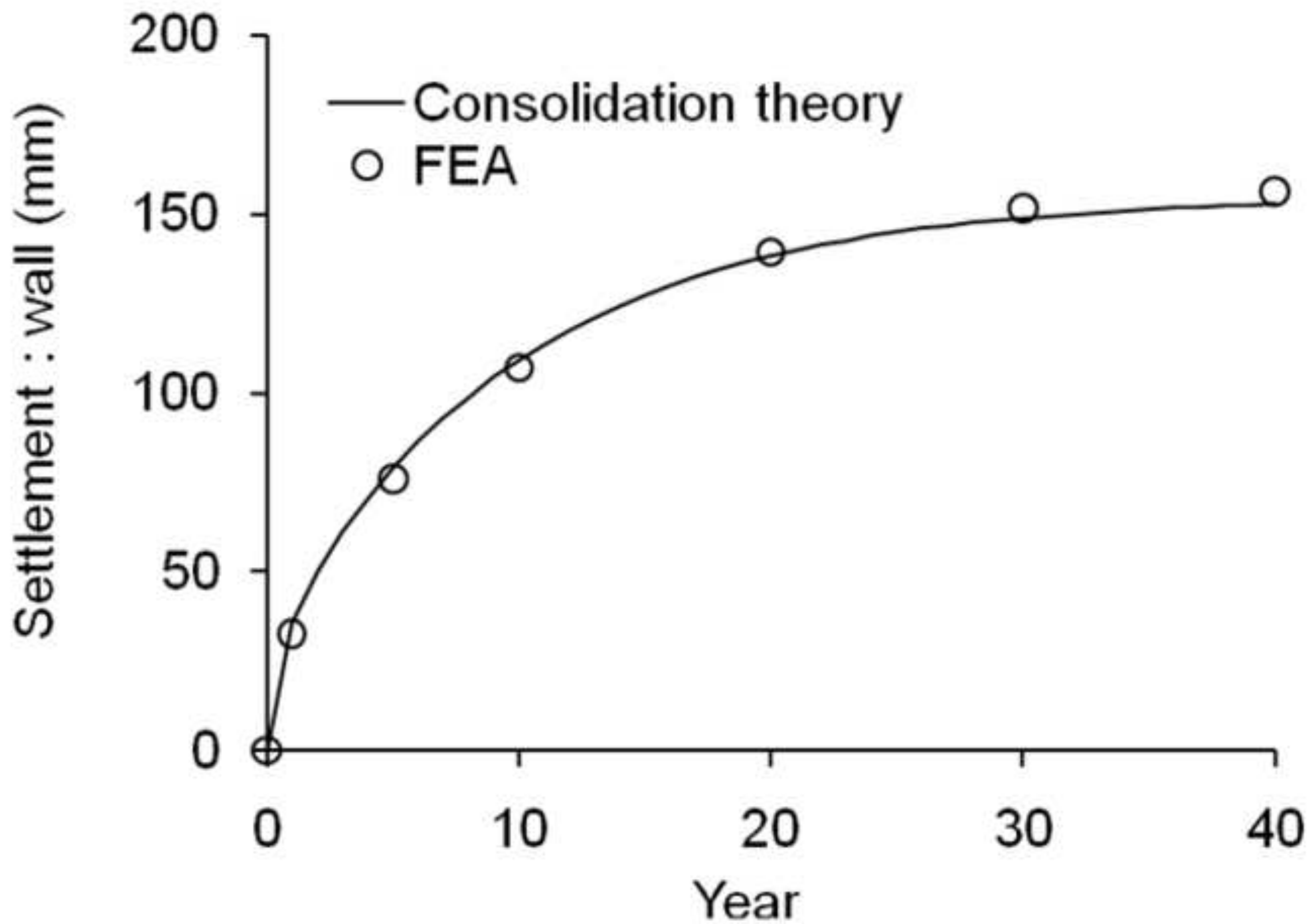

Figure 5a

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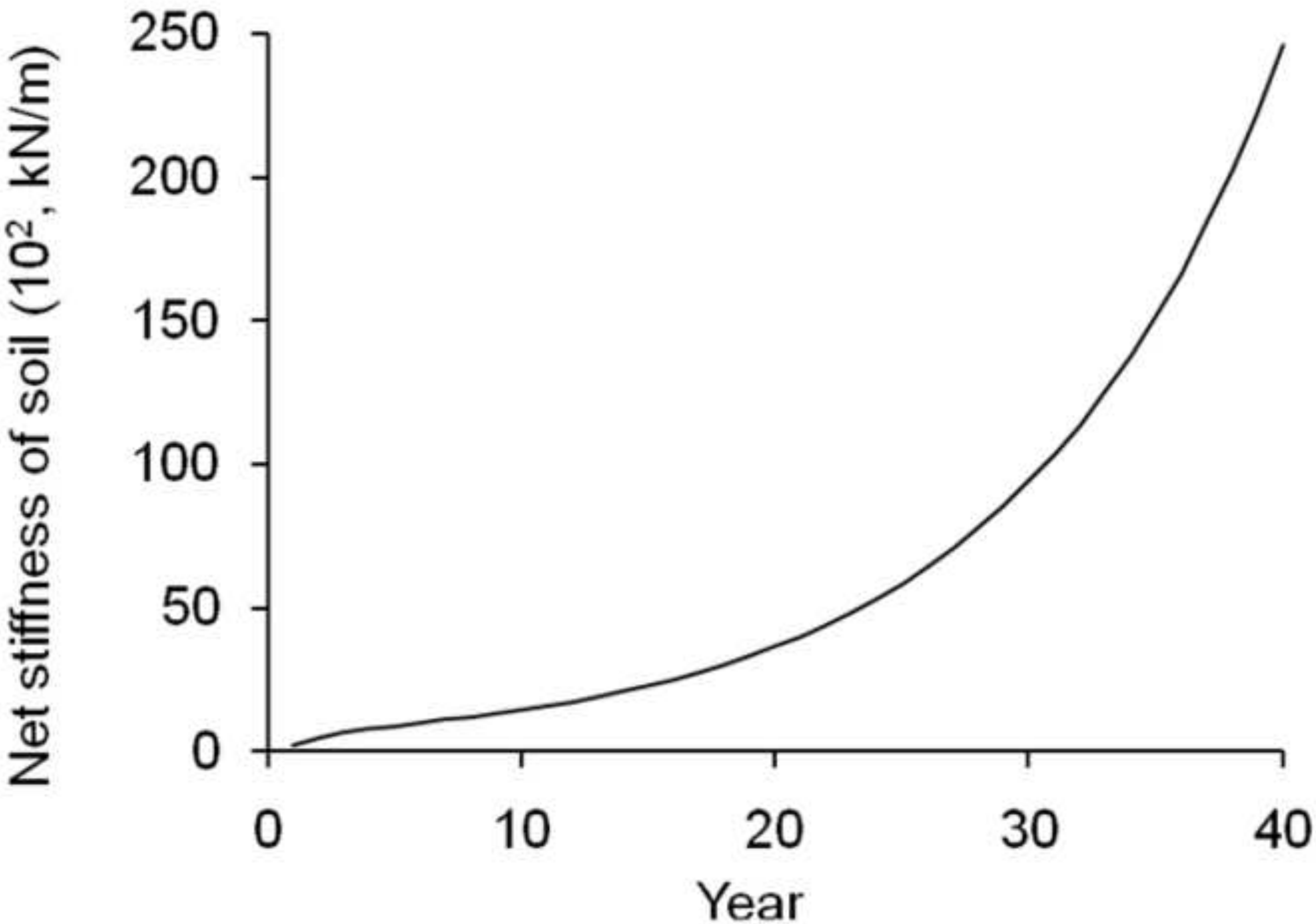

Figure 5b

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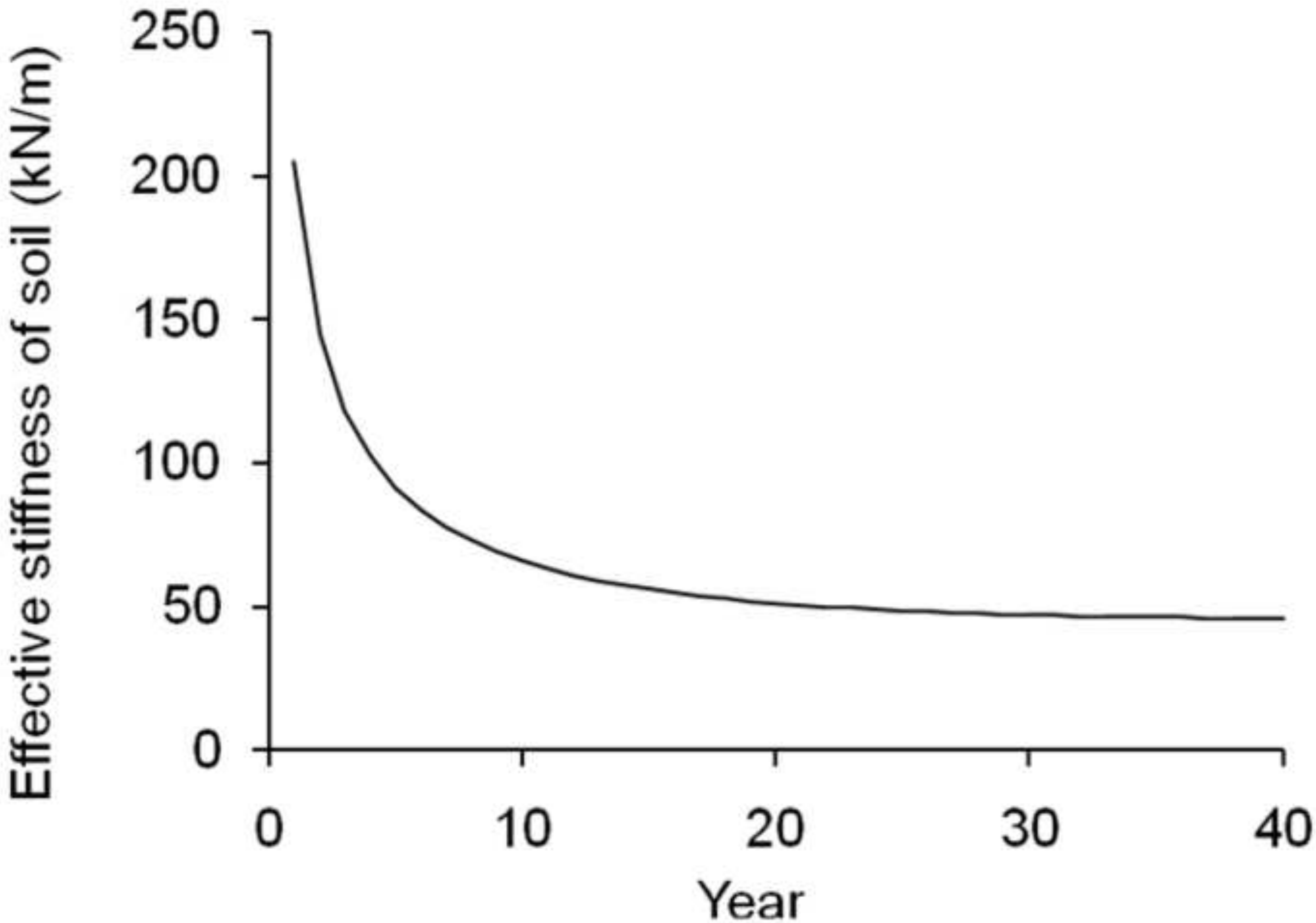

Figure 6a

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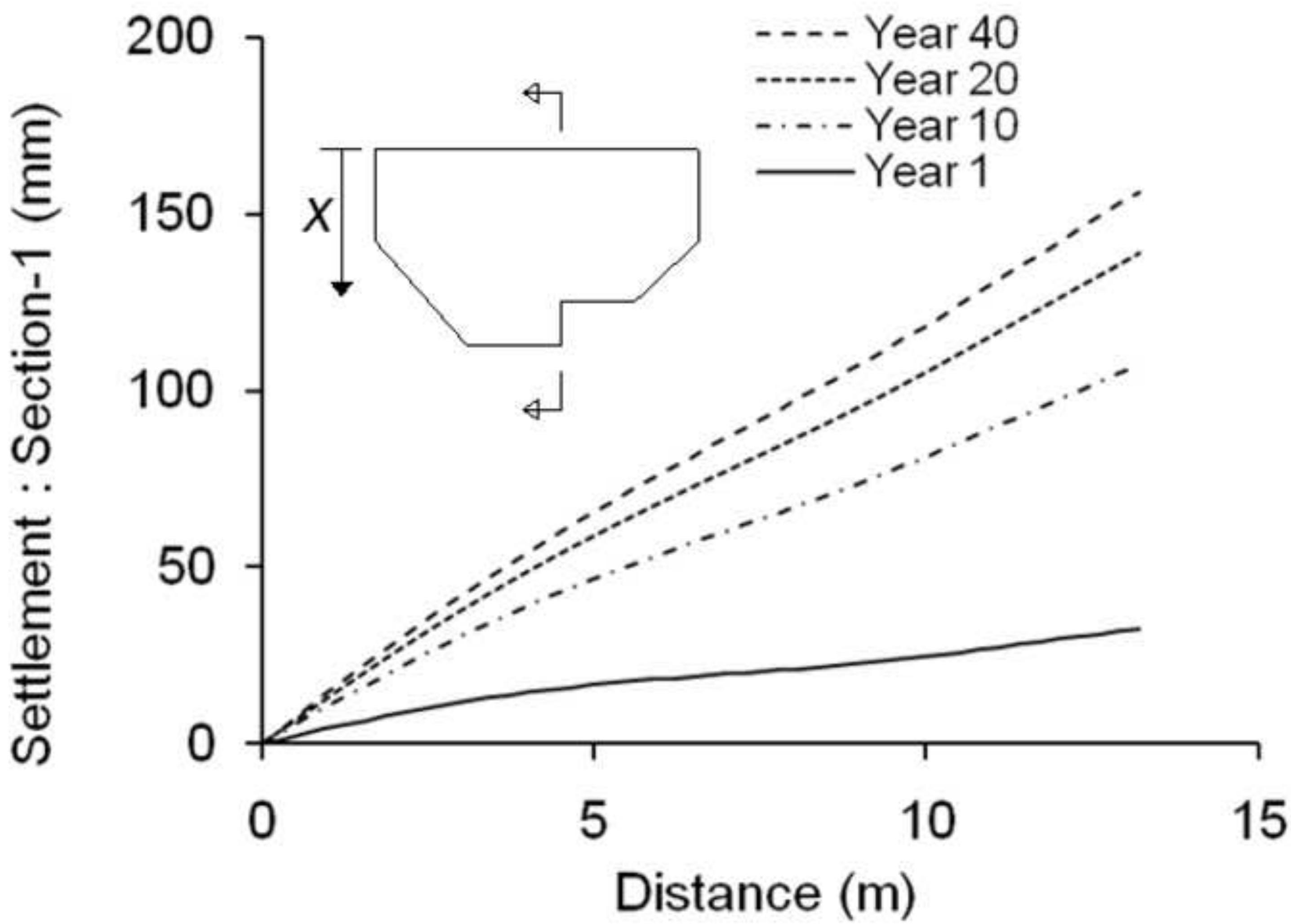

Figure 6b

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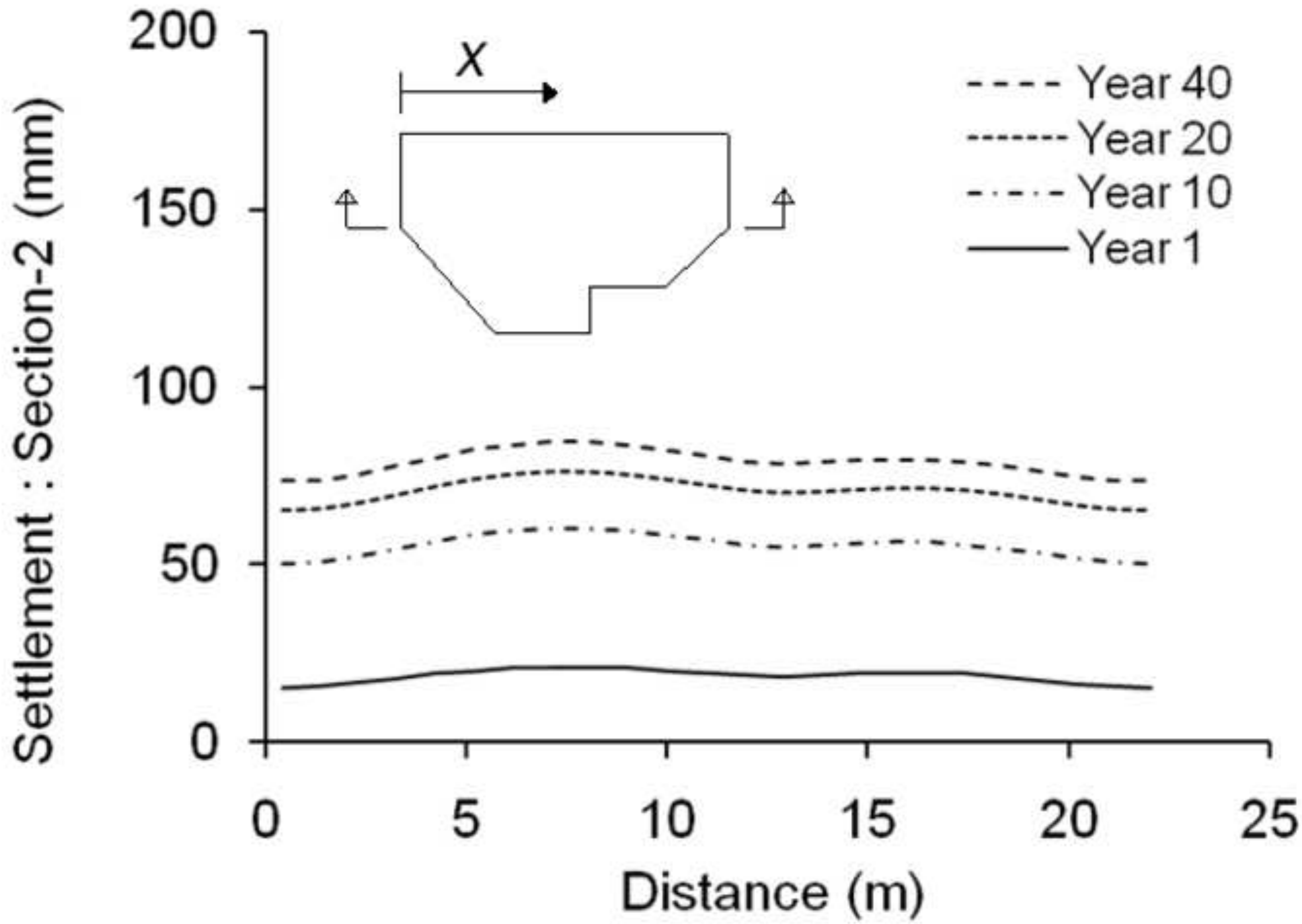

Figure 7a

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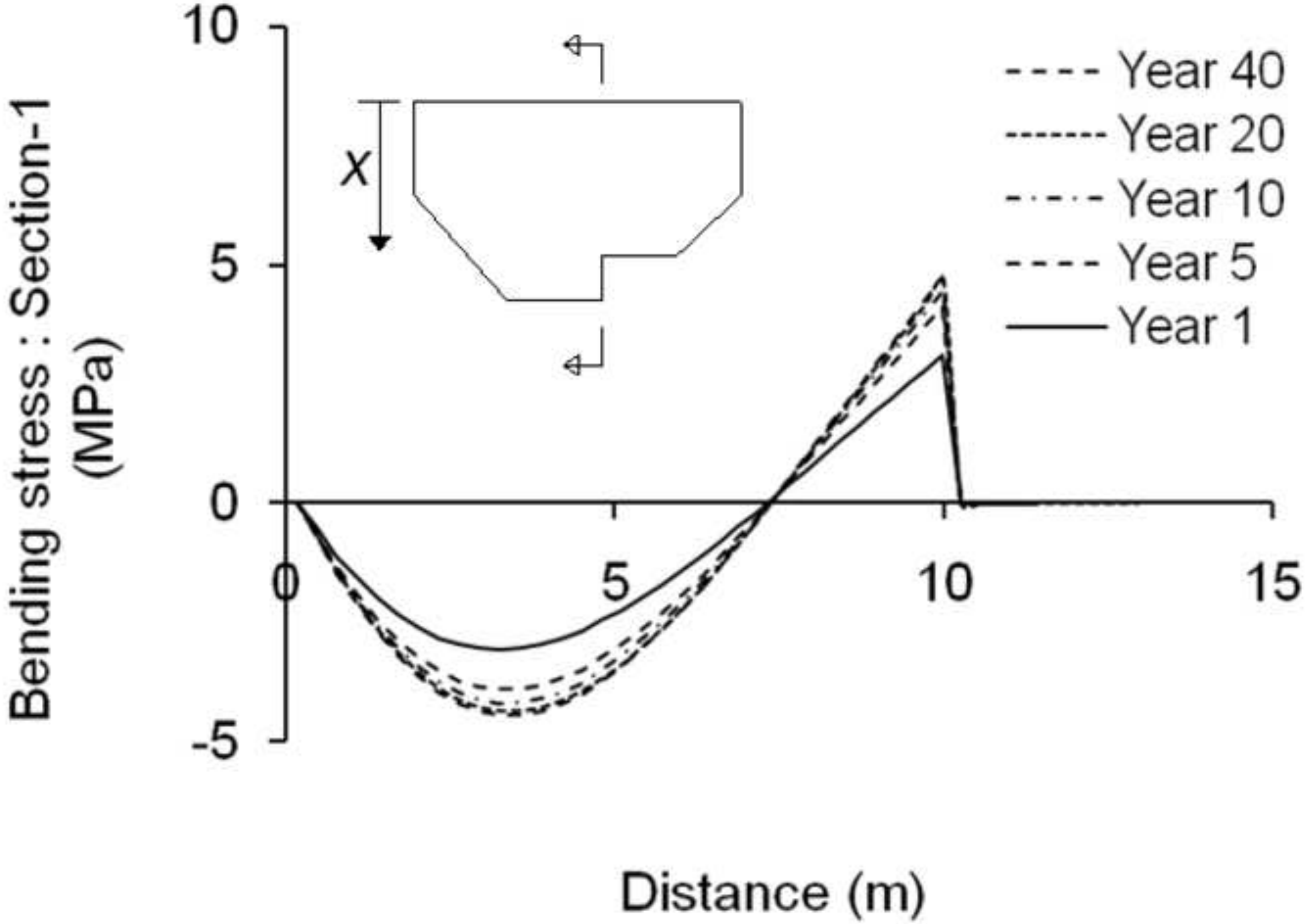

Figure 7b

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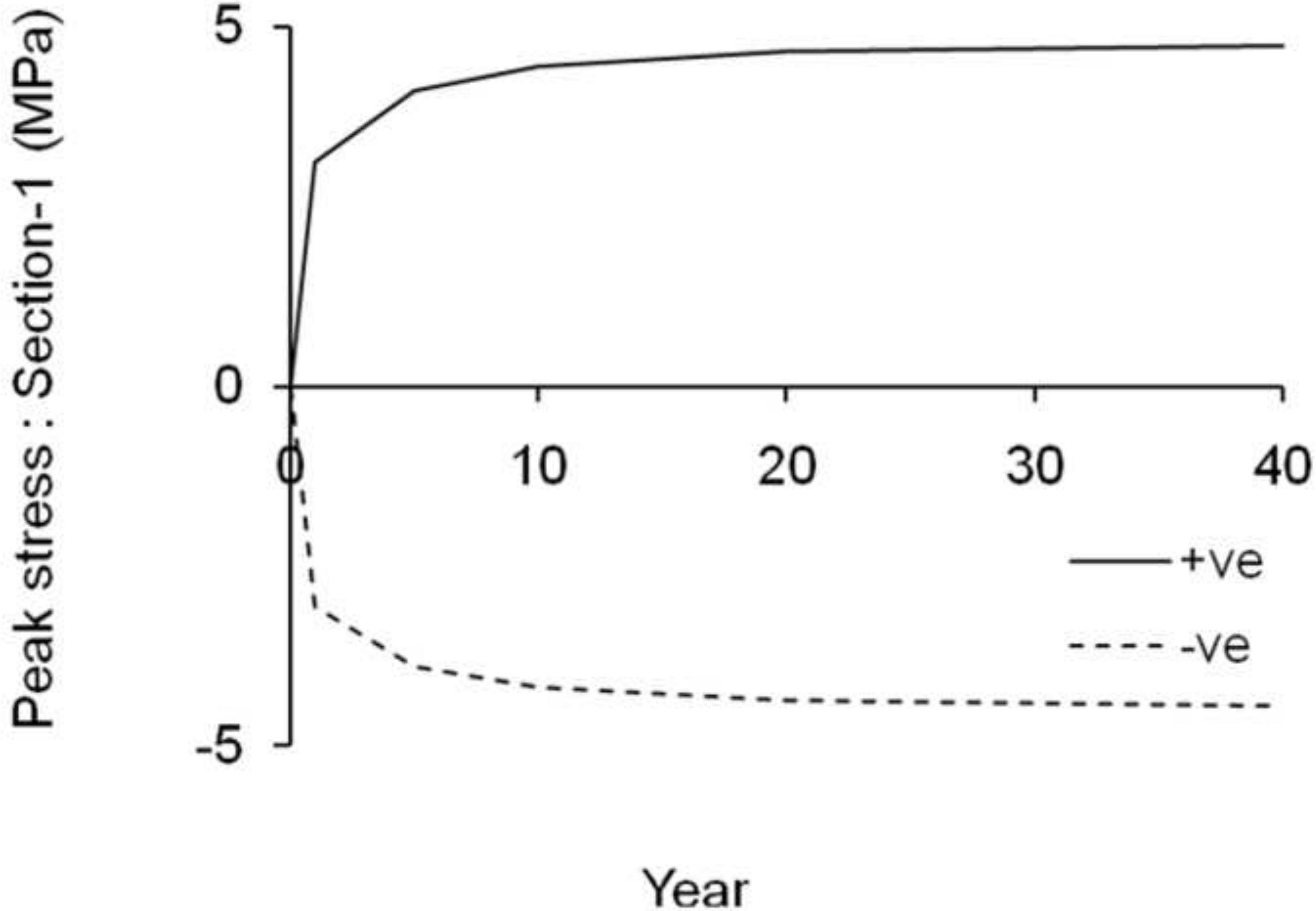

Figure 8a

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Figure 8b

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Figure 8c

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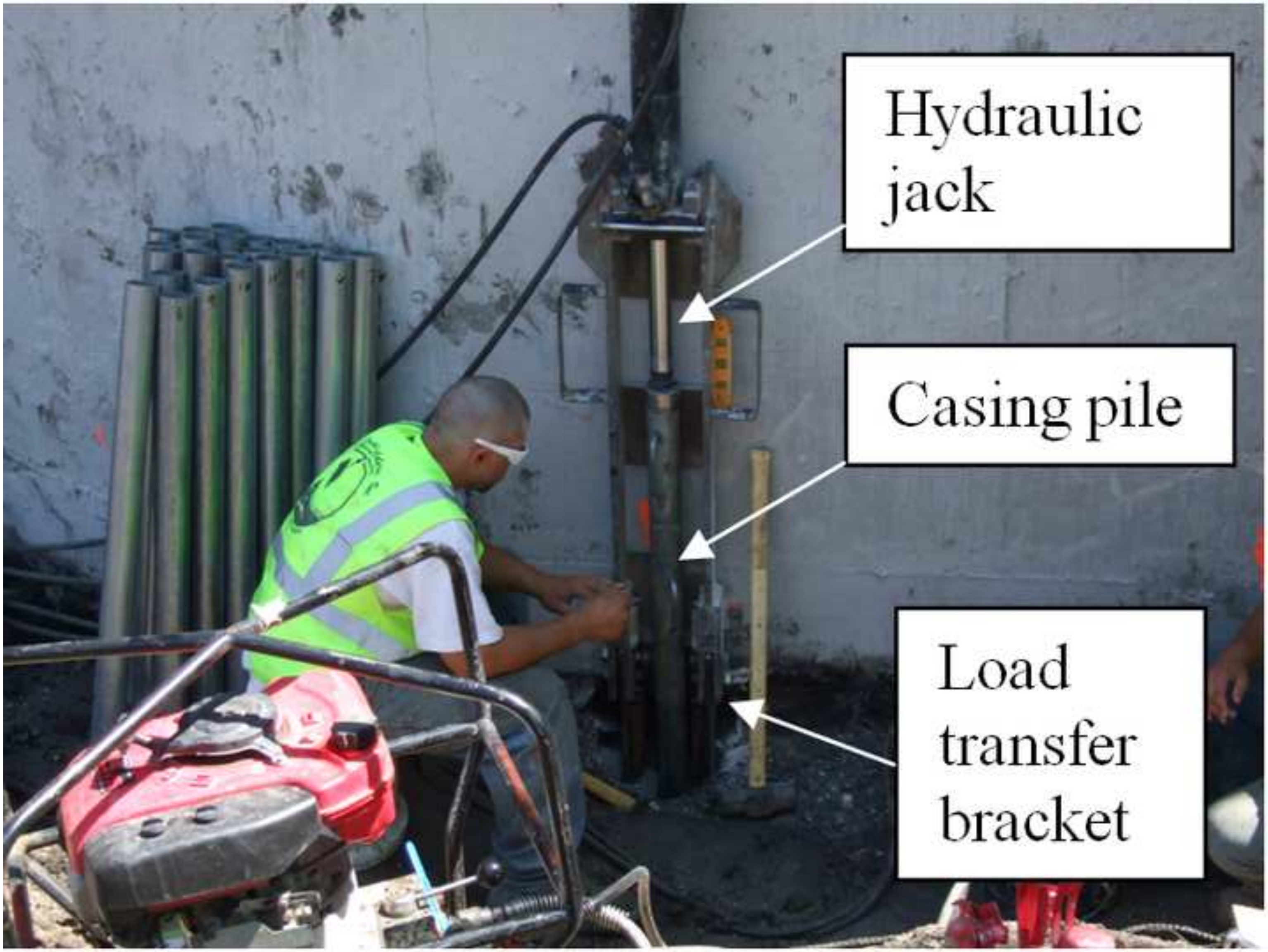

Figure 8d

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Figure 8e

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Figure 8f

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