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Predicting Reinforced Concrete Frame Response to Excavation Induced Settlement

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Abstract: In many tunneling and excavation projects, free-field vertical ground movements have been used to predict subsidence and empirical limits have been employed to evaluate risk. Validity of such approaches given the reality of two-dimensional ground movements and the influence of adjacent applied loads has been largely unknown. This paper employed analytical and large-scale experimental efforts to quantify these issues, in the case of a reinforced concrete frame structure adjacent to an excavation. Nearly half of all soil and building movements occurred prior to installation of the first tie-back, even when conservative practices were applied. Free-field data generated a trough half the size of that recorded near the building frames. Empirically-based relative gradient limits generally matched the extent and distribution of the damage. Application of various structural limits also generally reflected global experimental response but did not fully identify local damage distribution. Fully free-field data or failure to include accurate two-dimensional soil displacements under-predicted building response by as much as 50% for low-rise concrete frames without grade beams.

CE Database subject headings: Damage; Excavation; Reinforced concrete; Soil settlement; Subsidence.

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Introduction

At the onset of many large infrastructure projects, surface subsidence readings are taken in free-field or green field areas such as parks, in an attempt to quantify the volume and distribution of a settlement trough caused by tunneling or adjacent excavation prior to the application of those construction activities near buildings. These readings are then used to generate a generic settlement trough, based only on vertical settlement readings and an estimated horizontal component, which are then applied to the remainder of the project. Application of free-field measurements allows theoretical displacement, distortion, and strain to be estimated for all buildings situated wholly or partially within the predicted trough. The results are then compared to empirically based damage limits, or in the case of buildings of particular interest (e.g. protected heritage structures) used as inputs for finite element modeling. Yet how the nature of these free-field readings is altered by the combination of nearby applied loads and the validity of collecting and subsequently applying vertical data to predict building movement has not been fully understood. The following project combines the results of 1/10th scale experimental testing and numerical modeling to begin to quantify these issues.

Background

Over half a century ago, Skempton and MacDonald (1956) published their seminal work on allowable settlements, in which they established correlations between maximum angular distortion, maximum settlement, and maximum differential settlement. These were based on observations of 98 steel and reinforced concrete frame buildings, 40 of which experienced damage. Skempton and MacDonald collected their data predominantly from reported case studies. The ratio of differential settlement, δ, and the distance between two points, l (after eliminating the tilt influence of the building) were the basis of the damage criterion. The ratio δ/l was defined as the 'angular distortion', and limits were selected empirically at 1/300 for preventing cracks in walls and 1/150 for avoiding structural damage; these limits correspond to 2 and 4 cm respectively for typical 6 m spans. Furthermore, Skempton and MacDonald (1956) recommended that 'angular distortions' in excess of 1/500 should be avoided when possible, and that this should be decreased to 1/1000 to prevent all damage (1.2 and 0.6 cm respectively for 6 m spans). Subsequently Bjerrum (1963) supplemented Skempton and MacDonald's recommendations by relating the magnitude of δ/l to various types of damage. Some of the widely applied limits from the literature are summarized in Table 1.

Table 1. Failure criteria in terms of relative gradient

		Ultimate relative gradient limit			
Type of structure	Type of damage	Meyerhof (1947)	Skempton and MacDonald (1956)	Polshin and Tokar (1957)	Bjerrum (1963)
Frame structures and reinforced bearing walls	Structural	1/250	1/150	1/200	1/150
	Cracks in walls	1/500	1/300	1/500	1/500

Burland and Wroth (1975) investigated building damage due to foundation movement by considering the interaction between structures and their underlying soil. Main factors affecting interaction were identified as incrementally increased structural load, long-term soil consolidation, changing stiffness of the structure with deformation, and redistribution of loads and stresses within the structure due to differential soil settlement. They took as their starting point, the damage of cladding and finishes, rather than damage of structural members, because buildings usually become unserviceable before any danger of structural collapse emerges. As theoretical settlement predictions are directly dependent on Young's modulus (E), the influence of non-homogeneity of E was accentuated in that study. They plotted deflection limits for various ratios of E/G (effect of Poisson's ratio), where G is the shear modulus. Boone (1996 and 2001) examined 20 case histories and compared predicted results based on previous contributions, the introduction of a building's height to length ratio (an element not previously considered), and actual damage observations. Boone concluded that prior failure criteria were insufficient, because of the wide variety of building types. Recently, Finno et al. (2005) presented closed form solutions obtained via a laminated beam method as a departure from the previously developed deep beam model of Burland and Wroth (1975), for prediction of masonry building damage due to ground movements caused by excavations. Their model assumed that the floors restrain bending deformations, and the walls resist shear deformations. Confirmation was shown via a three-story framed structure affected by an adjacent excavation, for which they correctly estimated crack locations. Deflection of reinforced concrete members due to ground movement can be taken as a direct indication for damage prediction. As reported by Park and Paulay (1975), deflection limits (ratio of deflection of the member to its length) (Table 2) should not be exceeded for beams and one-way slabs that support or are attached to partitions likely to be damaged by large deflections.

Table 2. Maximum allowable computed deflections of structural members, (Park and Paulay 1975)

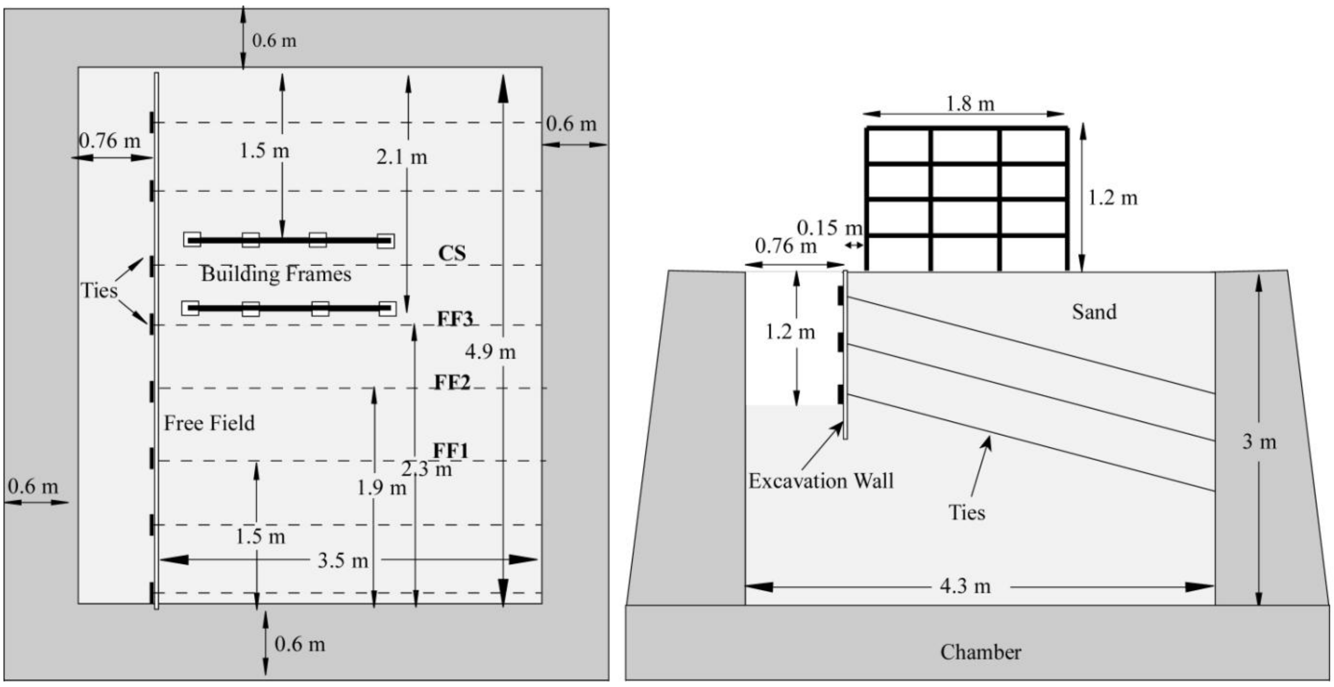
Member type	Deflection to be considered	Deflection ratio limit
Flat roofs not supporting or attached to non-structural elements likely to be damaged by large deflections	Immediate deflection due to live load	l/180
Floors not supporting or attached to non-structural elements likely to be damaged by large deflections	Immediate deflection due to live load	1/360
Roof or floor construction supporting or attached to non-structural elements likely to be damaged by large deflections		l/480
That part of the total deflection which occurs after the attachment of the non-structural elements, the sum of the long term deflection due to all sustained loads, and the immediate deflection due to any additional live load.	Roof or floor construction supporting or attached to non-structural elements not likely to be damaged by large deflections	l/240

Much of the aforementioned work is predicated on an assumption of a Gaussian distribution for tunnel and excavation induced ground movements. Hsieh and Ou (1998) proposed two settlement profiles: a spandrel to describe maximum settlement at a wall edge and a concave shape to depict maximum settlement setback away from a wall edge. To examine many of the abovementioned issues, a major laboratory program was undertaken, a portion of which is presented herein.

Experimental Program

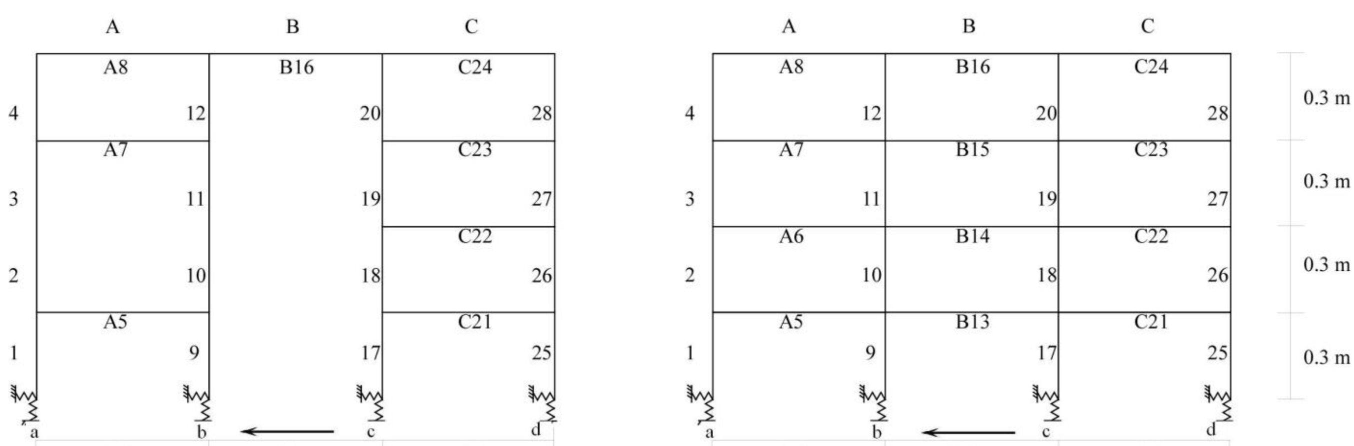
As part of that more comprehensive research program (Laefer, 2001), two large-scale tests (Test 2 and Test 6) were conducted to evaluate the response of excavation adjacent to reinforced concrete structures (Fig. 1). In each test, a pair of 1/10th scale concrete frames was set in dry sand perpendicular to a nearby excavation. Frames in each test were differentiated by East and West designations, depending on their location within the testing chamber. The frame buildings were four stories high (30 cm each) with three bays (60 cm each). In Test 2, the West building had a continuous beam at all levels except at the ground, while the Test 2 East (T2E) building had discontinuous beams, as in the case of a building with atrium spaces; omitting beams A6, B13, B14, and B15 (Fig. 2a). Test 2 West (T2W) and Test 6 West (T6W) buildings were identical to each other, with all beams present (Fig. 2b). All beam cross-sections were 3 cm x 4.6 cm, and column cross-sections were 4.6 cm x 4.6 cm. All frame dimensions, cross-sections, loads, and material properties were scaled to 1/10th of the prototype, in order to have code compliant models [UBC

1997 for loading and ACI 318-95 (ACI 1995) for member design] that also met scaling requirements (Laefer, 2001).



(a) Plan View (b) Cross-section

Fig. 1. Test setup (CS = Constructed-side instrumentation line, FF1, FF2, and FF3 = Free-field instrumentation lines)



(a) T2E (b) T2W and T6W

Fig. 2. Elements and dimensions of tested frames

Influence Zone of Frames

A Boussinesq analysis was employed using a line load assumption to consider the potential effects of the building on the free-field data collection zones. The applied load was 1.1 kN/m for the East buildings and 1.4 kN/m for the West buildings. In Fig. 3 the influence of both frames is shown superimposed. The testing scale was selected so that the applied dead load attenuated sufficiently at regions far from the frames to prevent the introduction of unwanted boundary condition effects. At the commencement of testing, all soil displacement readings were taken after the model buildings had been installed and loaded, akin to how measurements are made in the field. The zone between the buildings instrumented as shown as CS in Fig. 1 exhibits a high level of in situ stress, which diminishes significantly with distance from the buildings (instrumented as FF3, then FF2, and then FF1 as shown in Fig. 1). The influence of this in situ stress on post-loading displacements caused by excavation has yet to be quantified in the literature.

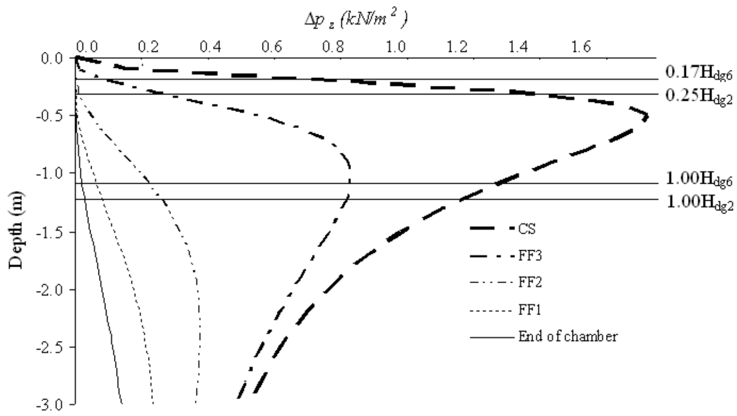


Fig. 3. Superimposed combined influence zone of the two frames in the test chamber on data collection locations with horizontal designators showing critical excavation levels for the two tests

Settlement Profile

In the featured study, soil was incrementally removed from in front of a three-level tied-back, continuous sheet piling wall in a series of lifts, and the tie-backs were loaded as the excavation proceeded (Fig. 1). Resultant soil settlement profiles are presented for two tests (Figs. 4 and 5) at two stages: immediately prior to the first tieback loading ($0.25H_{dg2}$ for Test 2, and $0.17H_{dg6}$ for Test 6) and at design grade ($1.00H_{dg}$). Settlement values (S_v) and distances from the retaining wall (d) were normalized by the respective final design grade, $1.00H_{dg}$. Design grade excavation depth was 122 cm for Test 2 ($1.00H_{dg2}$) and 109 cm for Test 6 ($1.00H_{dg6}$) as the soil in Test 6 was filled to only 89% of that of Test 2. This shorter soil profile influenced the system in several ways: (1) the first tie-back was installed earlier in the overall excavation cycle, thereby making the wall effectively stiffer, (2) less soil pressure was exerted against the whole wall, and (3) a shallower overall final design grade was excavated; each aspect theoretically contributed to less soil settlement. These depths are shown in Fig. 3. Frame displacements were measured at each stage of the excavation cycle (minimum of 15 steps). Although both Test 2 and Test 6 included a pair of loaded buildings, data from the Test 6 West was unavailable in adequate quantity to contribute to this analysis and was, thus, not presented herein.

The experimentally obtained settlement profiles (Figs. 4 and 5) showed similar displacement patterns for both the FF and CS. The closer the data recording positions were to the applied building loads, the greater the settlement, even though the measurement instruments were zeroed after the building frames were introduced into the testing chamber and loaded. At the $0.25H_{dg2}$ excavation level, the difference was more than double (1125 cm^3 vs. 533cm^3) and was 28% more at $1.00H_{dg2}$ (6355 cm^3 vs. 4993 cm^3). These ratios were similar for Test 6; the difference was 100% at $0.17H_{dg6}$ (329 cm^3 vs. 165 cm^3) and 49% (2181 cm^3 vs. 1462 cm^3) at $1.00H_{dg6}$. The large difference in the total volume of subsidence between the tests was a direct function of changes in wall geometry and excavation depth, as described above. In the more free-field sections (FF1 and FF2), during the $0.25H_{dg2}$ stage of Test 2 there was a frictional effect between the wall and the soil immediately behind the wall, thus accounting for the difference of recorded shape. This phenomenon (which would be termed as negative skin friction, if the subject was piling) does not occur in the areas closer to the frames (CS and FF3), where the superimposed load exerted by the foremost column footing further contributed to the settlement. Both recorded surface profiles reflected that predicted by Hsieh and Ou (1998) and were in agreement with the extent and shape proposed by Peck (1969) for installations of average workmanship in sand (Fig. 6).

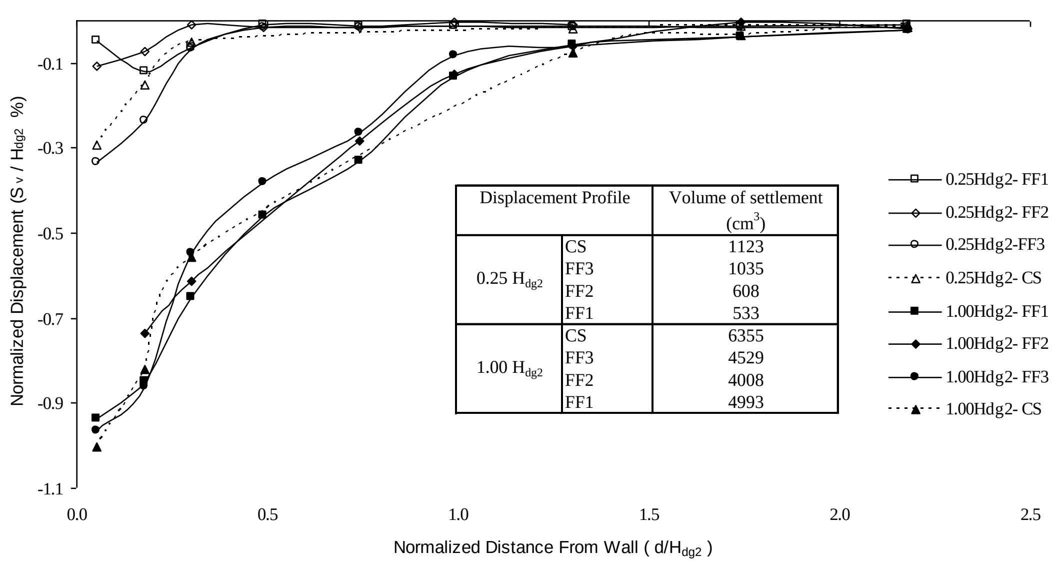


Fig. 4. Normalized vertical soil settlement with respect to normalized distance from excavation area (Test 2)

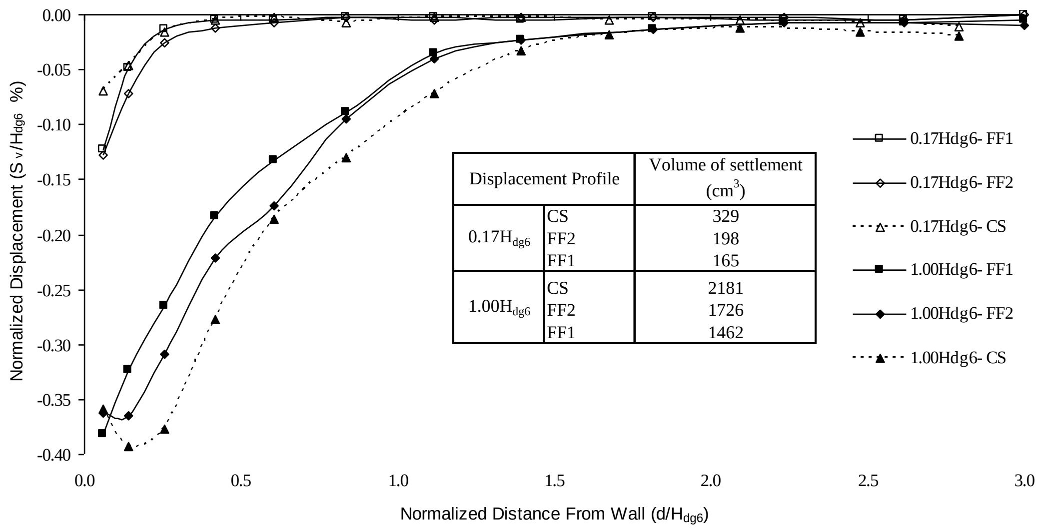


Fig. 5. Normalized vertical soil settlement with respect to normalized distance from excavation area (Test 6)

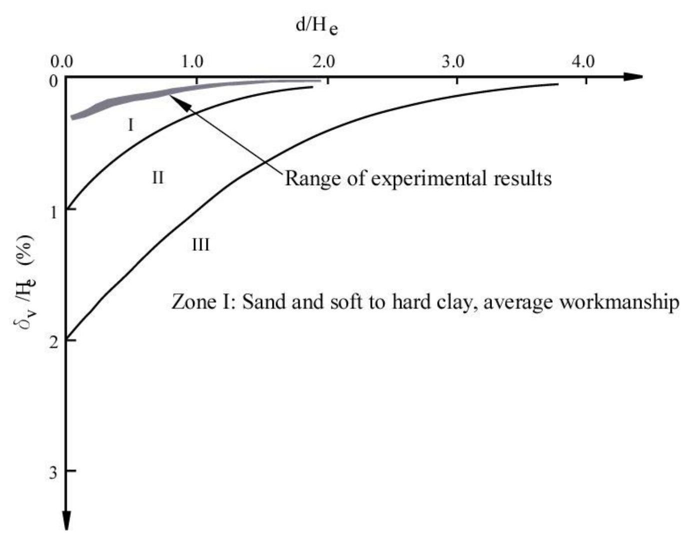


Fig. 6. Soil surface settlement (after Peck 1969)

Damage Approaches

In this study, analysis of soil settlement induced damage considered the following four topics:

Approach A: Shape and magnitude of the experimental settlement troughs at various excavation steps (just prior to tieback installation and design grade), with respect to only vertical displacement.

Approach B: Response of the frames at ground level versus recorded ground movements near to and far from the model buildings.

Approach C: Influence of horizontal input data and building stiffness with respect to structural code-based response.

Approach D: Structural member deformations and internal force generation versus structural damage criteria, with respect to computationally applied sets of soil settlement data.

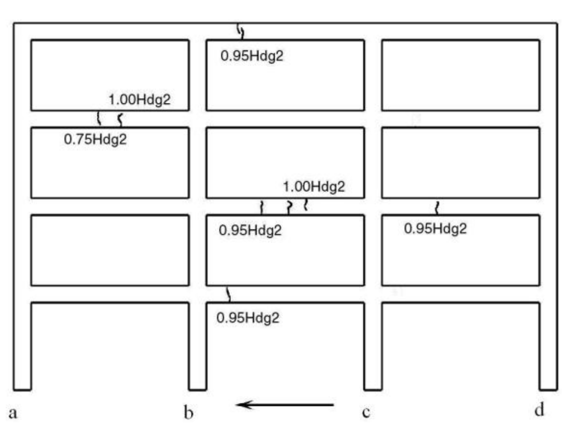
Approach A – Empirical Settlement Analysis

The 2D experimental displacements recorded at the base of the columns are shown in Table 3. Data not directly obtainable from specific instruments during experimentation were derived by linear interpolation of surrounding instruments. The maximum relative gradients between columns (calculated from deflections in Table 3) were 0.009 for T2E, 0.006 for T2W, and 0.004 for T6W. These exceeded the limits given in Table 1 and, therefore, were considered damaged, since the maximum allowable relative gradient is 0.006 for structural damage [Skempton and MacDonald (19560), and Bjerrum (1963)] and 0.003 for cracking [Skempton and MacDonald (19560)]. The actual locations and occurrence stages of the cracks in the frames experimented in this study are presented in Fig. 7 for T2W and T6W. There was generally good agreement between the gradient limits and the extent and distribution of the damage.

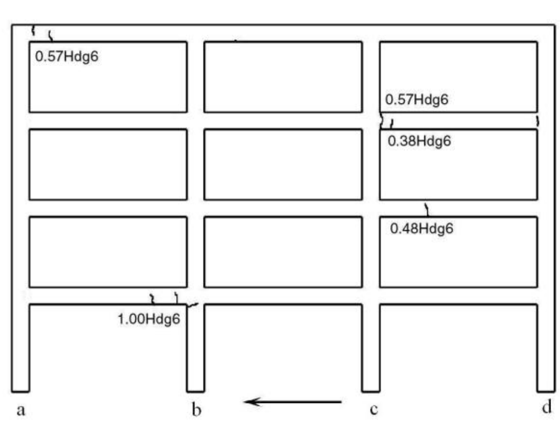
Table 3. Horizontal (x) and vertical (y) displacements of column foundations at final excavation stage (mm) and resulting maximum gradients

Test – Frame	Column a (x, y)	Column b (x, y)	Column c (x, y)	Column d (x, y)	Maximum Gradient between a and b	Maximum Gradient between b and c	Maximum Gradient between c and d
T2E	3.51, 10.59	6.25, 5.36	**3.65***, **2.78**	1.04, **0.20**	0.009	0.004	0.004
T2W	4.80, 10.72	4.01, 6.88	**3.49, 3.57**	2.97, **0.25**	0.006	0.006	0.006
T6W	1.19, 4.70	**1.00**, 2.46	0.81, 0.64	0.71, 0.08	0.004	0.003	0.001

*Bolded values were obtained by interpolation



(a) T2W



(b) T6W

Fig. 7. Locations and stages of structural cracks in the frames with directional arrow showing excavation position (note that only structural cracking was visible due to model size).

Approach B – Near Field and Far Field Input Differences, with Consideration of Horizontal Data Input

In engineering practice, predictive input data is usually obtained from a free-field, away from any structure. Numerical justification of such an approach has yet to be validated in the literature. Consequently a comparison between the results obtained from (i) *actual column displacements at their base (Col. Base)*, (ii) *construction side (CS) displacements, and (iii) free-field (FF) displacements* was made. The analytical models were created in the structural analysis modeling package Sofistik (V10.31-23). A total of 37 analytical runs were made across the 3 frames for multiple stages of excavation and tie-back installation, during the 2 experimental tests.

The experimental results (Table 4) showed that (1) soil surface settlement was generally less than that recorded at the base of the frame's columns, (2) data collected further from the frames documented smaller soil settlements, and (3) data at the end of excavation ($1.00H_{dg}$) was more consistent than that recorded early in the process ($0.25H_{dg2}$ and $0.17H_{dg6}$), because of instrumentation sensitivity. As such, critical questions arose about the appropriateness of applying free-field data to predict building response. To address this, in the numerical part of this study, model-scale RC frame structures (T2W, T2E and T6E) were subjected to the experimental soil subsidence profiles recorded at various offsets from the RC frames (as shown in Fig. 1), as well as the actual RC frame column base settlements (Table 4).

As the analysis was conducted on scaled meshes, the data obtained and reported in Table 5 were directly comparable to that measured from the $1/10^{th}$ scale laboratory model frames. No scaling or parameterization was applied. Table 5 shows that at the $1.00H_{dg}$ level, when the experimental displacements recorded at the base of the columns were used as input, the numerical results were generally within 10% of that experimentally recorded, but at the first tie-back installation level, the extremely small displacement levels did not appear to sufficiently consistent readings to achieve fully reliable prediction, because of instrumentation sensitivity.

Table 4. Input displacements employed for frame analyses (mm)

Test	Test step	Applied displacements	Column a		Column b		Column c		Column d	
			X	Y	X	Y	X	Y	X	Y
Test 2 East	0.25H$_{dg2}$	T2E-Col. Base-x y	0.25	1.68	0.41	0.28	**0.37***	**0.17**	0.33	**0.05**
		T2E-CS-x y	1.86	2.48	0.46	0.37	0.31	0.27	0.15	0.15
		T2E-FF 2-x y	0.95	1.53	0.27	0.27	0.24	0.24	0.09	0.05
	1.00H$_{dg2}$	T2E-Col. Base-x y	3.51	10.59	6.25	5.36	**3.65**	**2.78**	1.04	**0.20**
		T2E-CS-x y	1.30	11.00	0.50	4.60	0.33	1.80	0.20	0.40
		T2E-FF 2-x y	0.90	9.50	0.20	4.50	0.20	1.00	0.20	0.40
	0.25H$_{dg2}$	T2E-Col. Base-y only	-	1.68	-	0.28	-	**0.17**	-	**0.05**
		T2E-CS-y only	-	2.48	-	0.37	-	0.27	-	0.15
		T2E-FF 1-y only	-	0.80	-	0.20	-	0.20	-	0.20
		T2E-FF 2-y only	-	1.53	-	0.27	-	0.24	-	0.05
		T2E-FF 3-y only	-	2.80	-	0.30	-	0.20	-	0.20
	1.00H$_{dg2}$	T2E-Col. Base-y only	-	10.59	-	5.36	-	**2.78**	**-**	**0.20**
		T2E-CS-y only	-	11.00	-	4.60	-	1.80	-	0.40
		T2E-FF 1-y only	-	8.90	-	4.10	-	0.90	-	0.50
		T2E-FF 2-y only	-	9.50	-	4.50	-	1.00	-	0.40
		T2E-FF 3-y only	-	12.50	-	3.03	-	0.60	-	0.50
Test 2 West	0.25 H$_{dg2}$	T2W-Col. Base-x y	0.41	1.65	0.30	0.66	**0.15**	**0.41**	0.00	**0.15**
		T2W-CS-x y	1.86	2.48	0.46	0.37	0.31	0.27	0.15	0.15
		T2W-FF 2-x y	0.95	1.53	0.27	0.27	0.24	0.24	0.09	0.05
	1.00H$_{dg2}$	T2W-Col. Base-x y	4.80	10.72	4.01	6.88	**3.49**	**3.57**	2.97	**0.25**
		T2W-CS-x y	1.30	11.00	0.50	4.60	3.30	1.80	0.20	0.40
		T2W-FF 2-x y	0.90	9.50	0.20	4.50	0.20	1.00	0.20	0.40
	0.25 H$_{dg2}$	T2W-Col. Base-y only	-	1.65	-	0.66	**-**	**0.41**	-	**0.15**
		T2W-CS-y only	-	2.48	-	0.37	-	0.27	-	0.15
		T2W-FF 1-y only	-	0.80	-	0.20	-	0.20	-	0.20
		T2W-FF 2-y only	-	1.53	-	0.27	-	0.24	-	0.05
		T2W-FF 3-y only	-	2.80	-	0.30	-	0.20	-	0.20
	1.00H$_{dg2}$	T2W-Col. Base-y only	-	10.72	-	6.88	-	**3.57**	-	**0.25**
		T2W-CS-y only	-	11.00	-	4.60	-	1.80	-	0.40
		T2W-FF 1-y only	-	8.90	-	4.10	-	0.90	-	0.50
		T2W-FF 2-y only	-	9.50	-	4.50	-	1.00	-	0.40
		T2W-FF 3-y only	-	12.50	-	3.03	-	0.60	-	0.50
Test 6 West	1.00H$_{dg6}$	T6W-Col. Base-x y	1.19	4.70	**1.00**	2.46	0.81	0.64	0.71	0.08
		T6W-Col. Base-y only	-	4.70	-	2.46	-	0.64	-	0.08
		T6W-CS-y only	-	4.26	-	1.71	-	0.59	-	0.13
		T6W-FF 1-y only	-	3.71	-	1.20	-	0.31	-	0.13
		T6W-FF 2-y only	-	4.00	-	1.53	-	0.33	-	0.13

*Bolded values were obtained by interpolation

Generally, the results obtained by application of CS and FF3 data were +/-30% of those obtained by actual displacements, with most substantially under-predicting movements, particularly at the foremost column location. The results obtained with CS and FF3 displacements were substantially more similar to the results obtained from actual displacements (Col. Base), than those obtained when the FF1 and FF2 displacements were applied. For FF1 and FF2 input data, the results differed by more than 50%. Table 5 definitively showed that the closer to the frames the data was collected, the better the subsequent frame response matched the experimental results.

Table 5. Maximum displacements of analyzed frames with respect to applied soil profiles (mm)

Test	Test step	Applied Displacements	Story 1 X	Story 1 Y	Story 2 X	Story 2 Y	Story 3 X	Story 3 Y	Story 4 X	Story 4 Y
Test 2 East	$0.25H_{dg2}$	Experimental	NM*	NM	NM	NM	NM	NM	0.58	1.54
		T2E-Col. Base-xy	0.48	1.95	0.73	2.13	0.99	2.29	1.58	2.35
		T2E-CS-xy	0.29	0.53	0.40	0.72	0.53	0.88	0.65	0.94
		T2E-FF 2-xy	0.30	1.80	0.60	1.99	0.91	2.14	1.17	2.21
	$1.00H_{dg2}$	Experimental	NM	NM	NM	NM	NM	NM	8.28	12.40
		T2E-Col. Base-xy	3.15	10.85	4.94	11.04	6.88	11.18	8.73	11.25
		T2E-CS-xy	0.77	11.26	1.93	11.44	3.47	11.59	5.87	11.65
		T2E-FF 2-xy	0.71	9.76	1.71	9.95	2.83	10.09	3.93	10.16
	$0.25H_{dg2}$	T2E-Col. Base-y only	0.13	1.98	0.38	2.14	0.65	2.29	0.87	2.36
		T2E-CS-y only	0.20	2.75	0.55	2.93	0.94	3.09	1.25	3.15
		T2E-FF 1-y only	0.05	1.07	0.15	1.26	0.25	1.41	0.32	1.48
		T2E-FF 2-y only	0.13	1.80	0.37	1.99	0.63	2.14	0.85	2.21
		T2E-FF 3-y only	0.22	3.07	0.61	3.25	1.03	3.40	1.35	3.47
	$1.00H_{dg2}$	T2E-Col. Base-y only	0.69	10.86	2.15	11.04	3.79	11.19	5.38	11.26
		T2E-CS-y only	0.68	11.26	2.05	11.44	3.56	11.59	4.98	11.65
		T2E-FF 1-y only	0.46	9.16	1.33	9.35	2.32	9.50	3.28	9.56
		T2E-FF 2-y only	0.49	9.76	1.45	9.95	2.53	10.10	3.59	10.16
		T2E-FF 3-y only	0.83	12.75	2.37	12.92	4.02	13.07	5.42	13.13
Test 2 West	$0.25H_{dg2}$	Experimental	NM	NM	NM	NM	NM	NM	0.66	1.54
		T2W-Col. Base-xy	0.26	1.97	0.54	2.21	0.75	2.37	1.15	2.44
		T2W-CS-xy	0.74	2.79	1.17	3.03	1.51	3.18	2.04	3.25
		T2W-FF 2-xy	0.41	1.85	0.70	2.09	0.91	2.24	1.30	2.31
	$1.00H_{dg2}$	Experimental	NM	NM	NM	NM	NM	NM	9.68	10.24
		T2W-Col. Base-xy	4.39	11.04	6.03	11.28	7.68	11.44	9.56	11.51
		T2W-CS-xy	2.01	11.31	3.65	11.54	5.35	11.68	7.23	11.75
		T2W-FF 2-xy	0.90	9.81	2.27	10.04	3.70	10.20	5.32	10.26
	$0.25H_{dg2}$	T2W-Col. Base-y only	0.06	1.97	0.32	2.21	0.54	2.37	0.94	2.44
		T2W-CS-y only	0.12	2.80	0.51	3.03	0.86	3.18	1.39	3.25
		T2W-FF 1-y only	0.02	1.12	0.13	1.36	0.20	1.52	0.45	1.59
		T2W-FF 2-y only	0.06	1.85	0.32	2.09	0.54	2.24	0.93	2.31
		T2W-FF 3-y only	0.14	3.12	0.56	3.35	0.96	3.50	1.54	3.57
	$1.00H_{dg2}$	T2W-Col. Base-y only	0.65	11.04	2.21	11.28	3.87	11.44	5.76	11.51
		T2W-CS-y only	0.67	11.31	2.24	11.53	3.94	11.68	5.82	11.75
		T2W-FF 1-y only	0.52	9.21	1.76	9.44	3.10	9.60	4.60	9.66
		T2W-FF 2-y only	0.56	9.81	1.91	10.05	3.35	10.20	4.97	10.27
		T2W-FF 3-y only	0.77	12.79	2.55	13.01	4.49	13.15	6.59	13.21
Test 6 West	$1.00H_{dg6}$	Experimental	1.47	4.72	2.21	4.75	3.10	4.75	4.37	4.75
		T6W-Col. Base-xy	1.17	5.02	1.90	5.26	2.61	5.41	3.51	5.48
		T6W-Col. Base-y only	0.26	5.02	0.97	5.26	1.69	5.41	2.58	5.48
		T6W-CS-y only	0.24	4.58	0.87	4.81	1.52	4.97	2.34	5.04
		T6W-FF 1-y only	0.20	4.02	0.76	4.26	1.32	4.42	2.05	4.49
		T6W-FF 2-y only	0.22	4.32	0.82	4.55	1.42	4.71	2.19	4.78

*Not measured (NM) during the experiment

Approach C – Consideration of Horizontal Input and Building Stiffness with Respect to Structural Code-based Response

This part of the structural analysis involved further evaluation of input data. Some of the analysis employed as its input the *combined vertical and horizontal displacements* (described heretofore as 2D), while other runs had input restricted to *only vertical displacements* (described heretofore as 1D), which is the typical field-derived approach, because of the great difficulty in obtaining horizontal field data at ground surface.

Traditionally, although foundation settlement limits have been used to assess structural response to soil settlement, horizontal deflection limitations (inter-storey drift limits obtained by dividing the story drift by story height) of a superstructure are of considerable interest, in terms of both structural stability and building serviceability, because buildings vary greatly in terms of materials and geometry. Various structural codes and research provide insight on the relationship between horizontal building deformation and damage levels (e.g. Table 6).

Table 6. Performance level, damage state, and inter-story drift correlations (SEAOC, 1995)

Performance level	Damage state	Drift Limit (%)
Immediate occupancy	No damage	< 0.2
Damage control	Light damage	< 0.5
Life safety	Moderate damage	< 1.5
Limited safety	Severe damage	< 2.5
Collapse	-	> 2.5

Structural responses were considered with respect to inter-story drift ratios of the frames at each story level and compared to the code limits. The FEMA 356 maximum inter-story drift limit for RC buildings (where beams and columns are used as structural elements) is 0.003, for an immediate occupancy damage threshold (FEMA, 2000). The Uniform Building Code (UBC) provides a design limit under ultimate load and restricts the calculated story drift ratio to the smaller of $0.04/R_w$ or 0.005 for buildings less than 19.8 m, where R_w is a ductility reduction factor, and according to UBC R_w was 4 for frames examined in this study (UBC 1997). Therefore, the UBC limit considered herein was 0.005. The building was deemed damaged, if exceedance of these limits occurred at any story.

The influence of including the horizontal displacements was significant (Table 5). For the flexible frame T2E, the numerical results for first-story displacements differed greatly under each displacement profile. The 2D displacements generated 3.7 times more first-story drift than the 1D, when the excavation was at the $0.25H_{dg2}$ level of excavation, and 4.6 times than the 1D at design grade excavation ($1.00H_{dg2}$); these ratios attenuated at the upper stories to 1.7. Similar first-story behavior was also observed for T2W. The difference was 4.3 times at $0.25H_{dg2}$ and 6.7 times at $1.00H_{dg2}$, and it gradually decreased to 1.2 for $0.17H_{dg6}$ and 1.7 for $1.00H_{dg6}$, at the 4^{th} story; in soft story cases this effect may be especially critical. In Figs. 8-11 inter-story drift ratios of T2E, T2W, and T6W were plotted for the numerical results obtained by applying actual settlements and recorded settlements from Table 5.

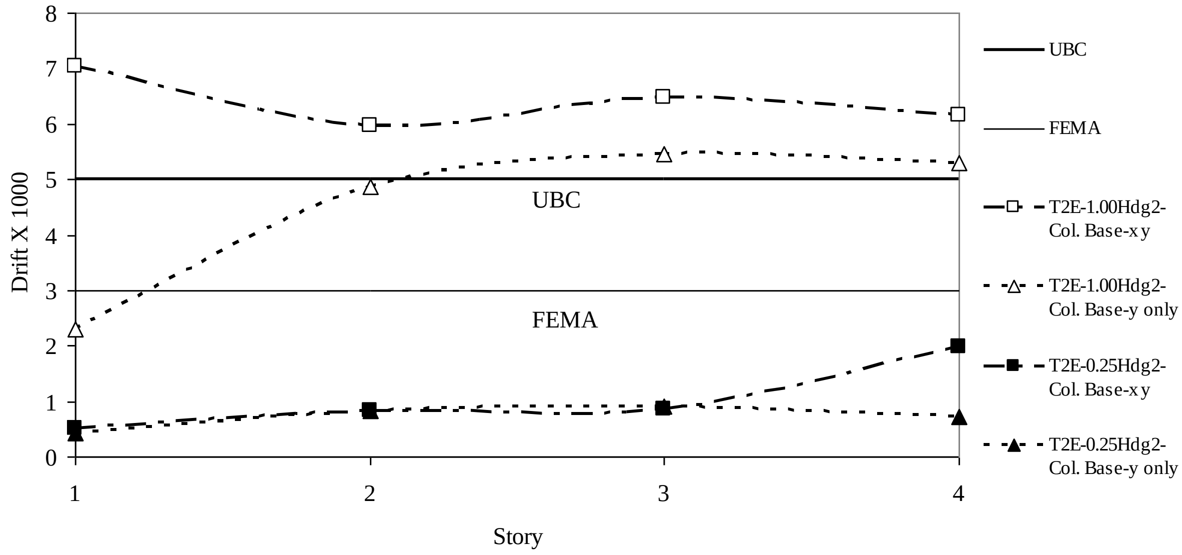


Fig. 8. Inter-story drift ratios of T2E at excavation levels of $0.25H_{dg2}$ and $1.00H_{dg2}$ with respect to 1D and 2D experimental column settlements

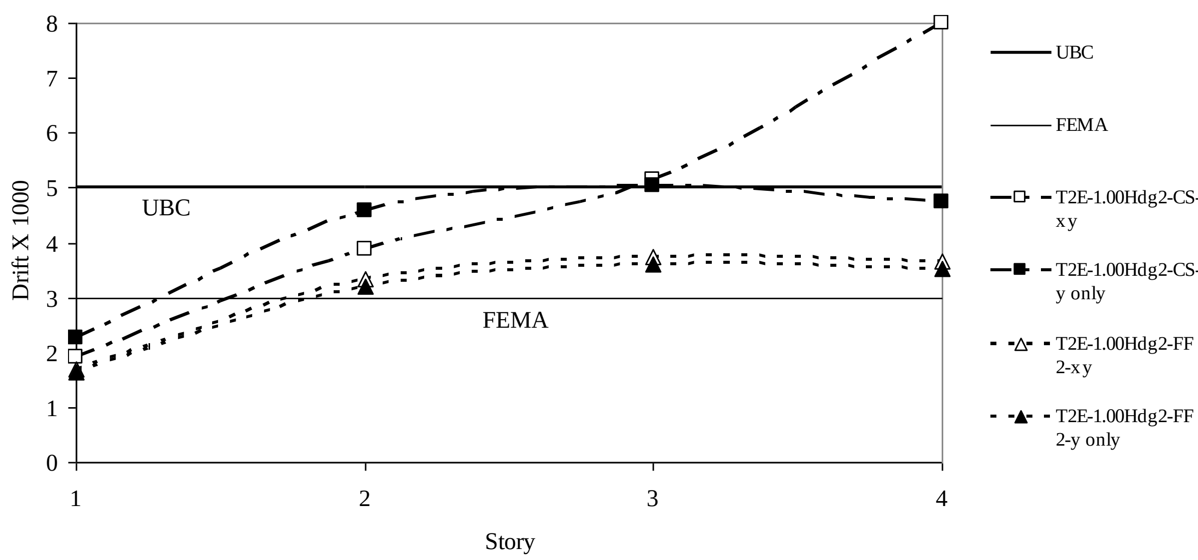


Fig. 9. Inter-story drift ratios of T2E at excavation levels of $1.00H_{dg2}$with respect to FF and CS soil settlements

The T2E inter-story drift ratios were above both of the damage limits at $1.00H_{dg2}$ but not at $0.25H_{dg2}$ (Fig. 8). In all cases, at 1.00 H_{dg2} the frame's response was more severe, when the horizontal component was included. This was most clearly shown at the first story, where interstory drift was severely under-estimated because of the frame's flexibility (200% in this story and 20% in upper stories). Frame T2E was further analyzed at $1.00H_{dg2}$ by applying FF and CS soil settlements (Fig. 9). Under these, the frame experienced inter-story drift ratios above the FEMA 356 limit, and in the case of the 2D CS input, the UBC design limit was also exceeded. Using data offset from the frames underestimated the drift by at least 15%, and even the inclusion of the horizontal data failed to produce the acute response shown in story 1 in Fig. 8.

Deformation profiles of T2W (Fig. 10) resulting from the application of actual column settlements were very similar to T2E, where high drift ratios exceeded code limits at the 1.00H$_{dg2}$ excavation level. These observations for T2W matched extremely well with the damage recorded on the model (Fig. 7). The difference in story 1 between T2W and T2E was because of the increased stiffness provided by the prevalence of beams in the former (Fig. 2).

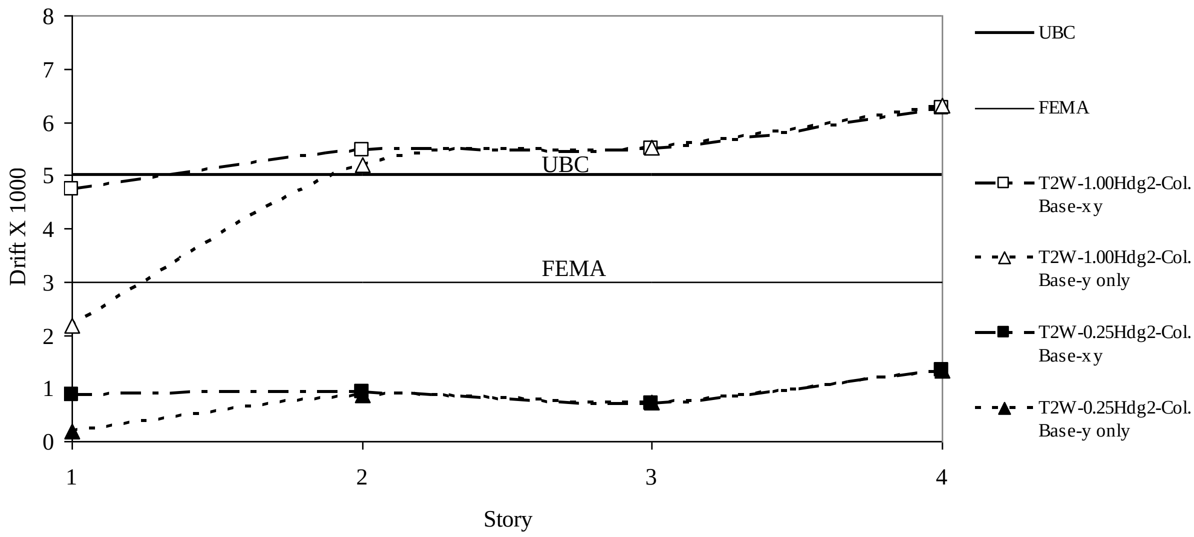


Fig. 10. Inter-story drift ratios of T2W at excavation levels of 0.25H$_{dg2}$ and 1.00H$_{dg2}$ with respect to 1D and 2D experimental column settlements

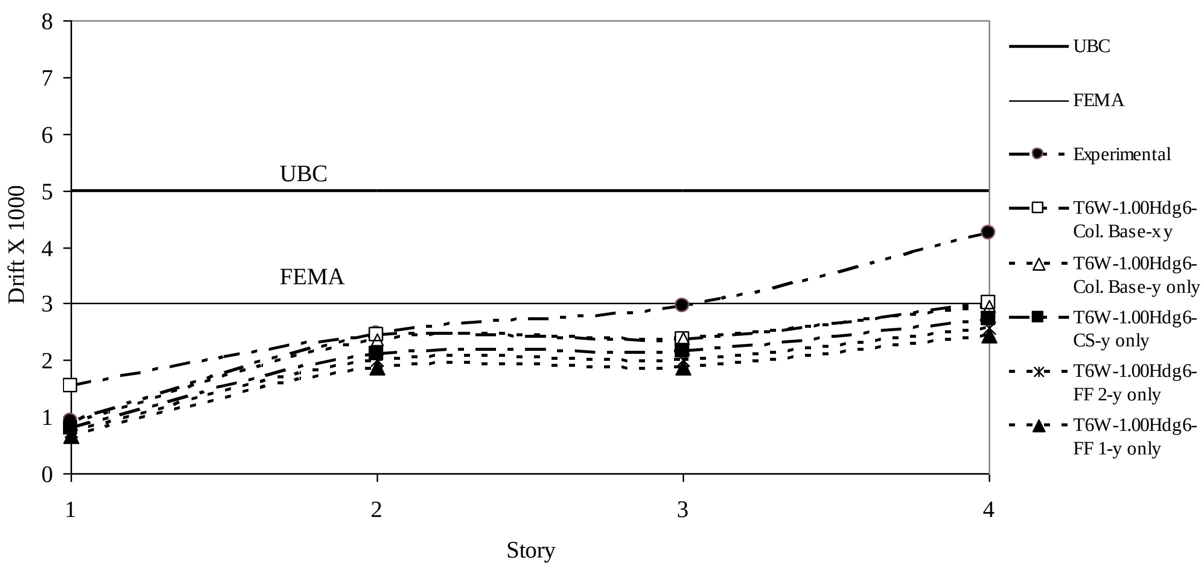


Fig. 11. Inter-story drift ratios of T6W at excavation level of 1.00H$_{dg6}$ subjected to near field and far field soil deformations

For frame T6W, experimentally measured drifts were available for this model, because of more extensive instrumentation (Fig. 11). The effect of horizontal ground displacements was observed again only at the first floor. Furthermore, the inter-story drift ratio patterns for all applied displacements were similar in shape but not in level, with a significant under-prediction for story 4. In this case, the difference was critical as it changed the damage designation with respect to the FEMA limit. Fig. 11 does less well predicting experimentally recorded damage (Fig. 7) than Fig.

10, which was more severe than predicted, but less than in previous tests. In summary, looking at the results of the 3 frames, the structural code limits agree well with the experimental results, where damage is significant but tend to under-predict in less severe cases.

Approach D – Assessment of Internal Member Forces and Beam Deflections

Long buildings aligned perpendicular to the excavation can be variously affected as the trough profile is non-linear. In such cases, detailed analyses may be warranted, to identify specific building portions that may be especially at risk. Tables 7 and 8 respectively show internal beam forces due to settlements at excavation levels of $0.25H_{dg}$ and $1.00H_{dg}$. Absent members in the T2E made this frame more flexible and, thus, more vulnerable to excavation settlements as it moved more compliantly with the soil than the more rigid frames. Most of the internal forces occurring in T2E exceeded those in T2W, and in several members, the ratio of the internal forces was more than double at both the $0.25H_{dg}$ and $1.00H_{dg}$ excavation levels. This was mostly attributable to the reduced number of members through which the forces could be distributed and the higher deformation levels.

Table 7. Frame member forces due to applied displacements at first tie-back installation level

Test	Member	Applied displacement profile					
		Col. Base - x y		CS - x y		FF 2 - x y	
		Moment (kNm)	Shear (kN)	Moment (kNm)	Shear (kN)	Moment (kNm)	Shear (kN)
Test 2 East	A5	0.09	0.30	0.11	0.53	0.06	0.29
	A7	0.06	0.21	0.13	0.42	0.07	0.23
	A8	0.06	0.17	0.10	0.32	0.05	0.18
	B16	0.03	0.14	0.05	0.21	0.04	0.24
	C21	0.03	0.09	0.06	0.20	0.03	0.09
	C22	0.04	0.12	0.07	0.16	0.04	0.13
	C23	0.03	0.11	0.07	0.14	0.03	0.12
	C24	0.02	0.05	0.04	0.12	0.02	0.05
Test 2 West	A5	0.07	0.24	0.16	0.68	0.10	0.38
	A6	0.04	0.15	0.12	0.39	0.07	0.23
	A7	0.04	0.15	0.11	0.41	0.06	0.24
	A8	0.04	0.12	0.10	0.34	0.06	0.20
	B13	0.01	0.04	0.07	0.28	0.04	0.15
	B14	0.02	0.07	0.05	0.18	0.03	0.13
	B15	0.02	0.08	0.06	0.22	0.04	0.15
	B16	0.02	0.07	0.04	0.20	0.03	0.13
	C21	0.03	0.09	0.09	0.29	0.04	0.12
	C22	0.02	0.05	0.04	0.15	0.02	0.06
	C23	0.02	0.07	0.06	0.19	0.03	0.08
	C24	0.02	0.05	0.04	0.13	0.02	0.06

The frames considered herein were assumed to be supporting non-structural elements (e.g. interior walls). Therefore, the allowable deflection limit for beam elements was l/480 (0.002) according to Table 2. Beam deflections for $0.25H_{dg}$ and $1.00H_{dg}$ excavation levels are presented in Tables 9 and 10, respectively. All deflections were in excess of the 0.002 deflection limit (serviceability), meaning they incurred at least some damage even prior to the installation of the first tie-back, but all structural damage occurred afterwards (Fig. 7). The predicted limits mostly matched the recorded damage. The effect of proximity of the structural elements to the excavation area is clearly seen in Tables 9 and 10, especially at $1.00H_{dg}$ level. The closest bay to the excavation is A and the furthest is C. The deflection ratios of the beams were generally in the same order: A >B > C. Examining the internal force distribution of the frames (Table 8), members of the bays closer to the excavation area were more loaded than those further away, similar to the member deflection ratios seen in Tables 9 and 10. In T2E and T2W, the deflections obtained from $0.25H_{dg}$ level excavation were at least half of those obtained at the $1.00H_{dg}$ excavation level. Thus confirming field experience showing that the excavation prior to installation of the first tie-back has a disproportionately high influence on the total behavior of an excavation system. Of note is that the results strongly concur with the input data (Table 4), where actual column displacement is less than recorded ground movement.

Table 8. Frame member forces due to applied displacements at design grade excavation level

Test	**Member**	**Applied displacement profile**					
		Col. Base- x y		**CS - x y**		**FF 2 - x y**	
		Moment (kNm)	**Shear (kN)**	**Moment (kNm)**	**Shear (kN)**	**Moment (kNm)**	**Shear (kN)**
Test 2 East	A5	0.51	1.05	0.30	1.17	0.27	1.05
	A7	0.19	0.70	0.22	0.74	0.20	0.71
	A8	0.15	0.48	0.16	0.53	0.11	0.39
	B16	0.07	0.23	0.11	0.22	0.08	0.17
	C21	0.04	0.10	0.14	0.22	0.09	0.32
	C22	0.12	0.36	0.13	0.41	0.13	0.46
	C23	0.11	0.39	0.12	0.41	0.13	0.48
	C24	0.06	0.22	0.08	0.28	0.12	0.43
Test 2 West	A5	0.16	0.60	0.23	0.90	0.21	0.88
	A6	0.04	0.13	0.26	0.93	0.17	0.62
	A7	0.03	0.12	0.23	0.87	0.16	0.60
	A8	0.03	0.09	0.21	0.71	0.12	0.45
	B13	0.04	0.15	0.14	0.21	0.09	0.29
	B14	0.01	0.05	0.06	0.23	0.07	0.21
	B15	0.01	0.05	0.04	0.22	0.05	0.16
	B16	0.02	0.05	0.03	0.21	0.06	0.14
	C21	0.02	0.02	0.13	0.23	0.15	0.54
	C22	0.01	0.04	0.14	0.59	0.19	0.67
	C23	0.01	0.04	0.17	0.62	0.20	0.72
	C24	0.01	0.03	0.13	0.48	0.16	0.59
Test 6 West	A5	0.10	0.39	0.23	0.90	0.22	0.88
	A6	0.06	0.22	0.27	0.93	0.17	0.62
	A7	0.06	0.21	0.24	0.87	0.16	0.60
	A8	0.04	0.16	0.20	0.70	0.15	0.45
	B13	0.05	0.19	0.14	0.21	0.10	0.30
	B14	0.04	0.11	0.06	0.23	0.07	0.21
	B15	0.03	0.10	0.05	0.22	0.06	0.16
	B16	0.03	0.08	0.04	0.21	0.06	0.14
	C21	0.07	0.24	0.13	0.23	0.15	0.54
	C22	0.07	0.26	0.15	0.59	0.19	0.67
	C23	0.08	0.29	0.18	0.62	0.20	0.72
	C24	0.07	0.24	0.18	0.48	0.16	0.59

Table 9. Beam deflection ratios at excavation level of $0.25H_{dg}$

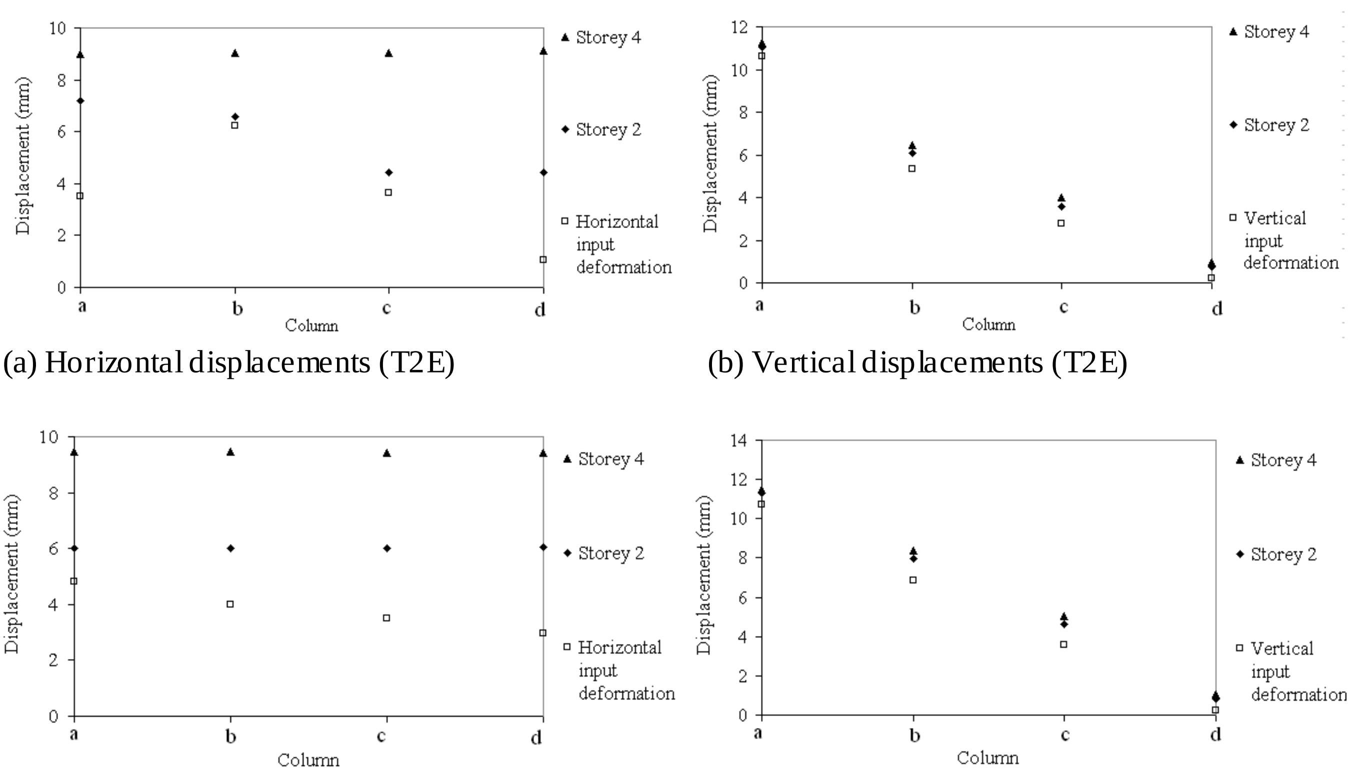
Test	Member	Applied displacement profile		
		Col. Base- x y	CS - x y	FF 2 - x y
Test 2 East	A5	0.006	0.008	0.006
	A7	0.006	0.007	0.006
	A8	0.005	0.006	0.005
	B16	0.004	0.004	0.004
	C21	0.005	0.003	0.005
	C22	0.005	0.005	0.005
	C23	0.005	0.005	0.005
	C24	0.004	0.004	0.004
Test 2 West	A5	0.005	0.007	0.006
	A6	0.005	0.006	0.005
	A7	0.005	0.006	0.005
	A8	0.004	0.006	0.005
	B13	0.004	0.004	0.004
	B14	0.004	0.004	0.004
	B15	0.004	0.004	0.004
	B16	0.003	0.003	0.003
	C21	0.005	0.004	0.005
	C22	0.005	0.005	0.005
	C23	0.005	0.005	0.005
	C24	0.005	0.005	0.005

Table 10. Beam deflection ratios at design grade, $1.00H_{dg}$, excavation level

Test	Member	Applied displacement profile		
		Col. Base- x y	CS - x y	FF 2 - x y
Test 2 East	A5	0.010	0.013	0.011
	A7	0.011	0.012	0.010
	A8	0.009	0.011	0.009
	B16	0.006	0.006	0.007
	C21	0.008	0.007	0.005
	C22	0.007	0.006	0.005
	C23	0.008	0.006	0.005
	C24	0.007	0.005	0.005
Test 2 West	A5	0.009	0.012	0.010
	A6	0.008	0.011	0.009
	A7	0.008	0.011	0.009
	A8	0.008	0.011	0.009
	B13	0.007	0.006	0.007
	B14	0.007	0.007	0.008
	B15	0.007	0.007	0.008
	B16	0.007	0.006	0.007
	C21	0.008	0.007	0.005
	C22	0.008	0.006	0.005
	C23	0.008	0.006	0.005
	C24	0.008	0.006	0.005
Test 6 West	A5	0.007	0.012	0.010
	A6	0.006	0.011	0.009
	A7	0.006	0.011	0.009
	A8	0.006	0.011	0.009
	B13	0.006	0.006	0.008
	B14	0.006	0.007	0.008
	B15	0.006	0.007	0.008
	B16	0.005	0.006	0.007
	C21	0.005	0.007	0.005
	C22	0.005	0.006	0.005
	C23	0.005	0.006	0.005
	C24	0.005	0.006	0.005

Soil Movement versus Building Movement

Comparing 2D soil displacements to the 2D building movements was the final consideration for analysis as it has implications for field monitoring. The relationship between the settlement occurring at the column foundations and at each story level was investigated by comparing these deflections in Fig. 12. Story deflections were monitored at the 2nd and 4th (top) stories. Although vertical deflections of the soil surface and both stories were similar, horizontal deflections differed significantly, because they were highly affected by rotation. The effect of beam continuity on horizontal movements was also observed. In Fig. 12 (a), the horizontal displacements of the 4th story did not change, unlike at the 2nd story due to the absence of a beam across each span. In Fig 12 (c), the horizontal displacements were largely uniform across each story but did not match the column base data because of rotation stories.



(a) Horizontal displacements (T2E) (b) Vertical displacements (T2E)

(c) Horizontal displacements (T2W) (d) Vertical displacements (T2W)

Fig. 12. Input displacements vs. story displacement for T2E-1.00H_{dg2}-Col. Base and T2W-1.00H_{dg2}-Col. Base displacement profiles

Similar to the soil settlement amount, nodal displacements experienced by the frames' bases were proportional to the proximity of structural elements to the excavation. The bottom storey experienced nonlinear deformation profiles because of the nonlinear trough shape, but at higher stories, the deformation became more uniform and linear. In Fig. 13, a typical example of vectorial deformations is shown, where response increased with proximity to the excavation.

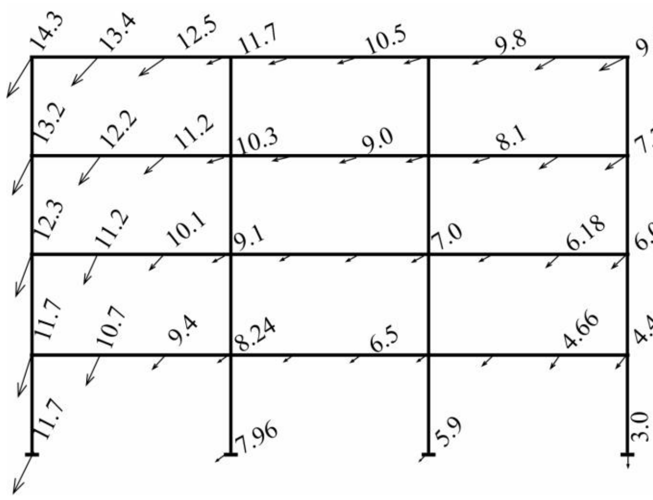


Fig. 13. Typical vectorial displacement patterns of T2W frame subjected to actual foundation displacements at $1.00H_{dg}$ excavation level (mm), (Sofistik V10.31-23)

DISCUSSION

Irrespective of which approach was selected (A-D), the frames were predicted to be damaged, to at least some degree, which was confirmed by inspection of the physical model (Figs. 7). Although empirical Approach A gave a strong idea about settlement vulnerability, it did not consider geometrical or material characteristics of the structures, and may mislead in the case of rigid frames with high stiffness characteristics. Therefore, application of possible settlement profiles to the numerical models provides clearer ideas of possible damage, because material and geometric considerations are incorporated. In Approaches B and C which relied upon finite element analysis, the importance of having representative input data, with respect to horizontal ground movements and the influence of existing applied loads was shown clearly using interstorey drift limits. Where input data did not clearly match actual column movement, damage was under-predicted, often significantly. In many cases obtaining such representative data may create substantial challenges and compensatory adjustments may need to be made to prevent under-predicting damage levels. Application of inter-story drift limits also tended to under-predict damage levels. The same was true to a lesser extent by applying beam deflection limits (Approach D), which provided more detailed information on building behavior including rotation and displacement of individual members. With this type of analysis, at-risk portions of a structure could be monitored and possibly be preventively reinforced, although the system was better at global performance prediction than local damage prediction.

CONCLUSIONS

One-tenth scale laboratory work investigating the response of reinforced concrete frames to adjacent excavation induced settlement was combined with numerical modeling to determine the most appropriate set of input parameters and evaluation criteria based on standard field measurement approaches in both the geotechnical and structural engineering communities. The results were validated for low-rise RC frame structures without grade beams.

- A disproportionate amount of both soil settlement and building displacement, often more than 50%, occurred prior to the installation of the first tie-back even when this occurred at 0.17% of the final excavation level and the total prototype excavation was only 10.9 m.
- Measured soil trough displacement near the building frames was substantially higher than in the free-field zone (as much as 100% prior to the first tie-back installation and up to 50% at design grade).
- Empirical gradient limits generally correlated with recorded damage levels and distributions
- The closer the data was collected to the structure, the better the numerical displacement prediction was, with up to 50% difference, when fully free-field data were applied.
- Inclusion of horizontal displacements increased first story drifts by 3.7–6.7 times depending upon the frame and testing stage and can, therefore, be considered extremely important for risk evaluation, especially where a soft first story exists or a structure is relatively flexible.
- Using fully free-field soil data severely under-predicted inter-story drift levels, even when horizontal displacements were applied, and in many cases, changed the predicted damage level to a less acute one.
- Use of inter-story drift levels was not fully applicable, where experimental damage was recorded, even though design cut-off limits were not fully reached. Slightly better agreements were seen using design deflection limits established for members supporting non-structural elements.
- Depending upon frame geometry, displacement induced forces were twice as much in the flexible frame as the more rigid frame.
- According to deflection ratio limits for members supporting non-structural elements all beam deflections exceeded damage limits prior to the installation of the first tie-back.

- The vertical displacements experienced by columns are representative of vertical soil displacements at all stories, but horizontal ones are not due to rotation, especially where buildings have more flexible sections.
- Various structural evaluation methods were considered with respect to inter-story drift, beam deflection, and force levels. Although additionally potentially highly useful information was obtained full correlation with experimental damage was not achieved, and that global performance prediction was better than local member damage identification.

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