International terminal building chevron expansions Vancouver international airport, British Columbia

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Abstract: The east and west chevron expansions to the International Terminal Building (ITB) at Vancouver International Airport are founded on deep, soft, compressible deltaic deposits located in an area of high seismic activity. The site subsurface conditions include loose sand pockets that are potentially liquefiable under a major earthquake. The function of the ITB and its chevron expansions required large clear spans, therefore, the building structures required large steel moment frames. While the ITB and east chevron expansion consisted of 3-storey buildings with narrow underground baggage tunnels, the proposed west chevron expansion included a heavy aquarium display area as well as a large underground baggage hall area. This paper presents the main considerations involved with site preparation and building foundation design for the ITB chevron expansions. The design of shallow foundations subjected to a wide range of vertical, horizontal, and moment loads as well as large long-term post-construction settlements required an extensive interaction between geotechnical and structural engineers. This paper emphasizes the importance of such "human soil-structure interaction" in foundation engineering design.

Geotechnical Aspects

Site and Subsurface Conditions

Vancouver International Airport is located within the central portion of Sea Island, Richmond, British Columbia (Fig. 1). Sea Island is bordered by branches of the Fraser River on the northeast and south sides and the Strait of Georgia along the west side. The natural grade of the island is part of a large delta that has been built up over several thousand years. The general pattern of sediment deposition has followed the direction of Fraser River flow from east to west, and has built up soft sediments of about 200 m thickness at Sea Island (Luternauer, et al. 1993, Mathews and Shepard 1962).

Figure 2 shows the layout of the International Terminal Building (ITB) and its chevron expansions and the location of relevant test holes within the site. The east and west chevron expansions extend eastward and westward from the north end of the ITB. The building envelope for the east chevron measures about 200 m by 55 m, with the northeast edge expanding into a bell shape towards the surrounding east apron area. The building envelope for the west chevron measures about 360 m by 55 m, with the northwest edge also expanding into a bell shape towards the surrounding west apron area.

The original grade in 1992 prior to the existing ITB development was at about geodetic elevation 1.1 m with a grass and topsoil cover (Meyerhof and Sebastian 1970). The existing grade within the concrete apron surrounding the ITB chevron expansions is at about geodetic elevation 1.80 m.

Fig. 1 Vancouver international airport key plan

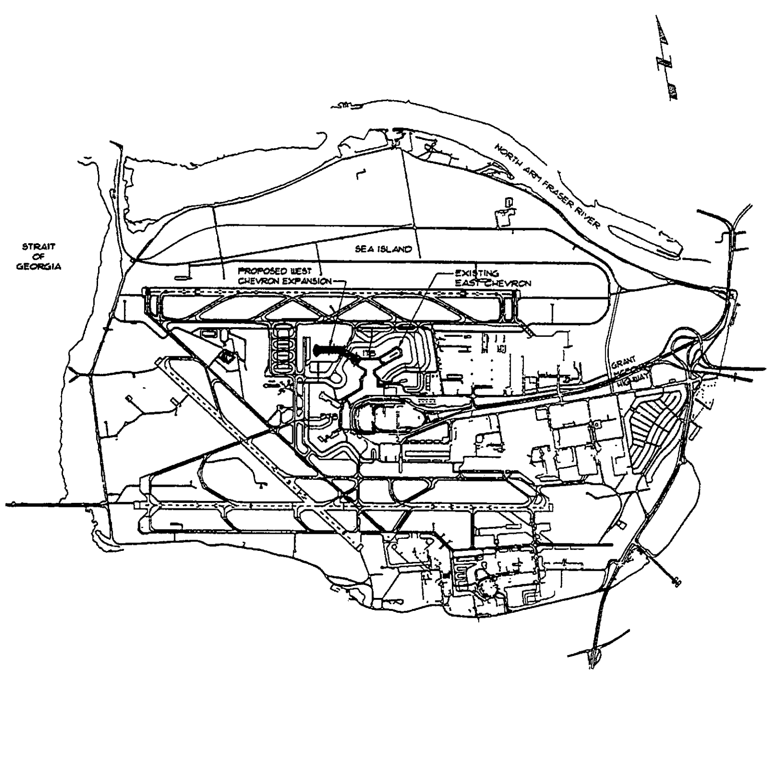
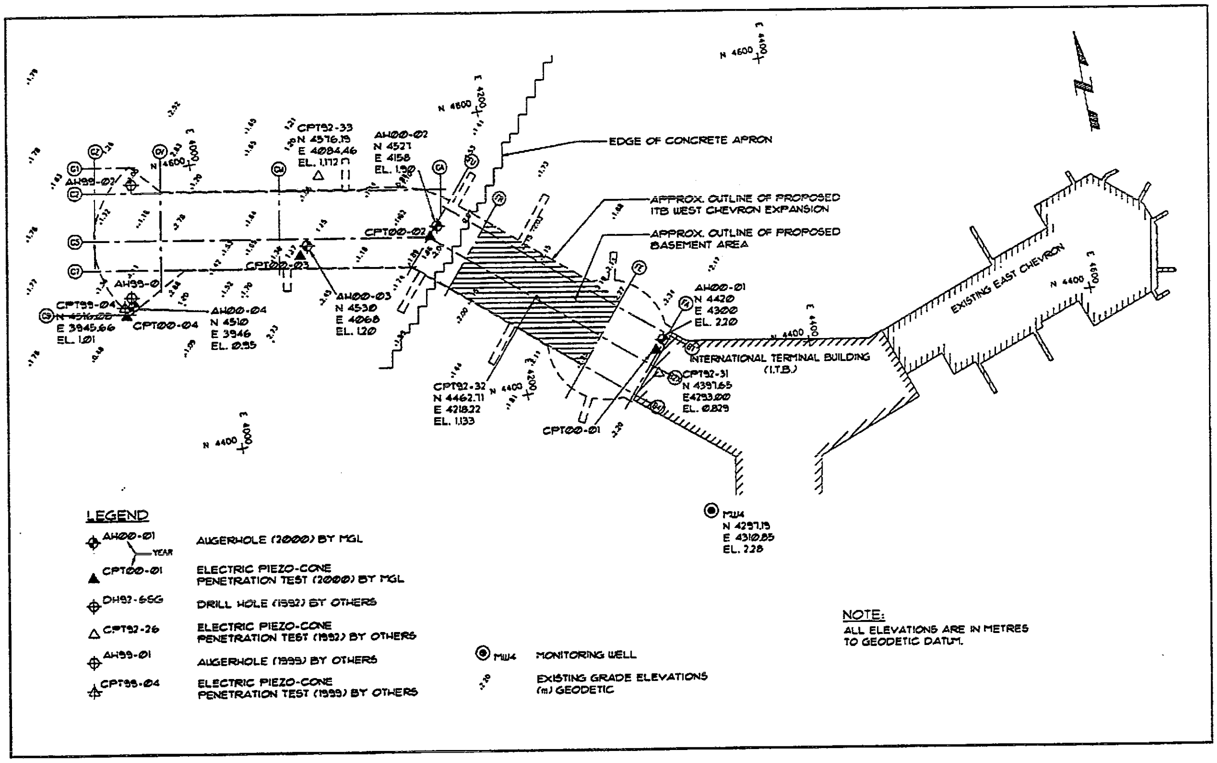


Fig. 2 ITB chevron expansion layout and test holes



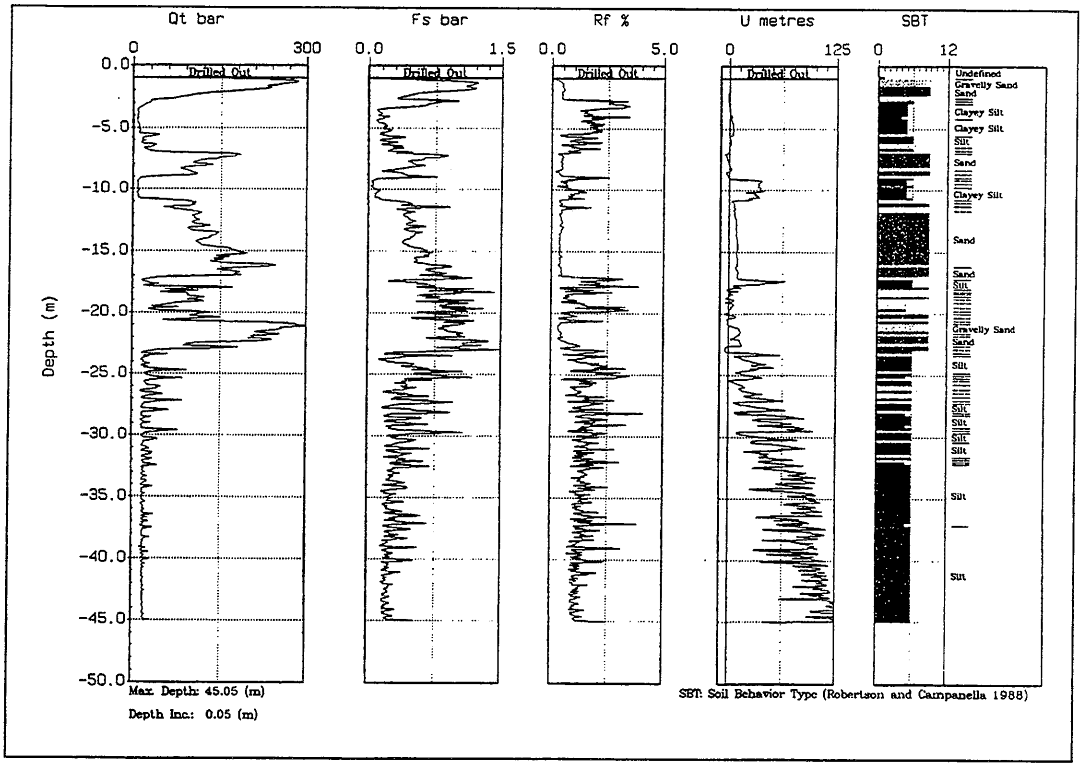
Recent test hole data within the ITB chevron expansions indicated that a generalized subsoil stratigraphy would include in order of increasing depth (Fig. 3):

- Zone 1: Granular fill layer.
- Zone 2: Clayey silt, trace of organics, becoming sandy SILT in bottom 1 m (estimated thickness 3 to 4 m).
- Zone 3: Sand, inter-layered with sandy silt/silty sand in the upper 5 m, loose to dense with depth (estimated thickness 10 to 20 m).
- Zone 4: Inter-layered sand and clayey silt (estimated thickness up to 8 m).
- Zone 5: Silt to clayey silt (estimated thickness 130 m).

Zone 1 underlying the existing apron and road structure consisted of a grey-brown, well-graded, compact, medium to coarse, clean sand of about 0.7 m thickness. Generally, this granular fill consisted of 75 mm minus Sechelt sand or pumped Fraser River sand with less than 8% passing the U.S. standard sieve no. 200.

Zone 2 consisted of a brown to grey, firm to stiff, clayey silt with occasional organic inclusions with a thickness ranging from 3 to 4 m at the time of investigation. The test hole data indicated that this clayey silt stratum had a natural water content ranging from 36 to 54% with a liquid limit of about 58%, and a plastic limit of about 33%, indicating a soil of medium plasticity. The undrained shear strength of the clayey silt zone was estimated from the CPT measurements. In the east portion of the proposed west chevron expansion, the undrained strength ranged from 40 kPa to 55 kPa with an average of about 45 kPa. In the west portion of the site, the undrained strength ranged from about 30 kPa to about 50 kPa with an average of about 40 kPa. The above range of undrained shear strength indicated that the soil consistency within this zone was firm where thumb can penetrate the soil to stiff where thumb can indent the soil. Generally, previously preloaded portions of the site resulted in a higher average undrained strength that in unfilled areas of the site. However, a clear relationship between undrained strength and preload height could not be identified.

Fig. 3 Typical subsoil stratigraphy



In non-preloaded areas, this soil zone was considered to be of medium compressibility with a relatively fast settlement response time to loads in excess of natural overburden pressure. The quick response to load within this zone was due to its short drainage path and limited thickness. The monitoring of settlements under preload fills for nearby sites indicated that this soil zone could undergo both primary and secondary forms of subsoil settlement. The primary settlement was often completed within 4 to 6 weeks of load application whereas secondary settlement could be on-going for years but at a much smaller rate, which varied with soil thickness, natural water content, plasticity index, and extent and thickness of fill load added above natural grade.

Zone 3 consisted of a grey, well-graded, silty sand of about 10 to 20 m thickness. The bottom of the sand zone is estimated at about geodetic elevation -13 to -25 m. This subsoil zone included mostly loose sand with soft silty portions varying from 1 to 2 m in thickness and medium dense sand portions ranging from 1 to 2 m in thickness. The fine-grained soils in this zone were considered to have a low compressibility and a relatively fast settlement response time to loads in excess of natural overburden pressure.

Zone 4 consisted of a grey, inter-layered fine sand, silt, silty sand, and sandy silt layers. The estimated thickness of zone 4 varied from 6 to 8 m. This inter-layered silt/sand stratum was loose to compact, had a water content varying from 32 to 40%, a liquid limit of about 33%, and a plastic limit of about 26%. This soil zone was considered to be of medium to high compressibility with a relatively moderate settlement response time to loads in excess of natural overburden pressure due to the presence of numerous sand lenses and partings.

Zone 5 consisted of grey, clayey silt with occasional thin silt and sand layers. This stratum extended to a depth in excess of 100 m. Available information from deep geological boreholes near Sea Island suggested that this clayey silt stratum could extend down to bedrock at a depth of about 240 m. The clayey Silt stratum of zone 5 had average water contents of about 33 to 35% with a liquid limit of about 35%, and a plastic limit of about 30%. This zone was considered to be of medium to high compressibility with a relatively slow settlement response time to loads in excess of natural overburden pressure due to its long drainage path and extensive thickness.

Available records of measurements taken before the ITB construction activity in 1993 indicated groundwater levels ranging from geodetic elevation -1.2 to 1.2 m. Long-term data from monitoring wells near the north end and west side of the ITB indicated that groundwater levels ranging from geodetic elevation -1.14 to +0.58 m from September 1996 to September 1997. Additional groundwater monitoring indicated water levels varying between geodetic elevation -0.63 m and +0.06 m from March 1998 to December 1999.

The highest groundwater level was measured in March 1999 and was about 1.74 m below the concrete apron existing grade.

Seismic Considerations

The Fraser River Delta and Sea Island areas are located in a high seismic zone due to the stresses induced in the underlying bedrock by the collision of the offshore Pacific plate with the American continental plates. According to the seismic model of the 1995 National Building Code of Canada (NBCC), the peak ground acceleration (PGA) and peak ground velocity (PGV) for firm ground or bedrock surface for different risk scenarios are as given in the following table.

Table 1 Seismic Risk from the 1995 NBCC Seismic Model

Probability of Occurrence per year	0.01	0.005	0.0021
Return period (years)	100	200	475
PGA (g)	0.09	0.14	0.22
PGV (m/s)	0.08	0.12	0.22

The predominant seismic risk for the 475 years return period was related to earthquakes of magnitude 6.5 to 7.5 with distances to the epicentre ranging from 25 to 100 km. The NBCC located the Vancouver International Airport within the seismic zone with $Z_a = Z_v = 4$ with respect to firm ground acceleration and velocity. The thick soft soil deposits overlying the bedrock in the Fraser River delta tended to increase the predominant period of the bedrock ground motion.

To allow for this effect, it was recommended that (Byrne et al 1991):

- The design earthquake be of magnitude 7.0;
- For geotechnical design aspects, the surface ground acceleration be increased from 0.21g to 0.30g; and,
- For structural design aspects, a foundation factor of F equal to 2 be used.

The results of liquefaction analysis carried out down to a 30-m depth based on available CPT data and the above seismic design parameters indicated that liquefiable Sand pockets extended from geodetic elevation -1.0 to –14 m within the site. These potentially liquefiable sand pockets were limited to the site areas that have not been densified previously by vibro-replacement stone columns. The following comments apply only to these non-densified areas.

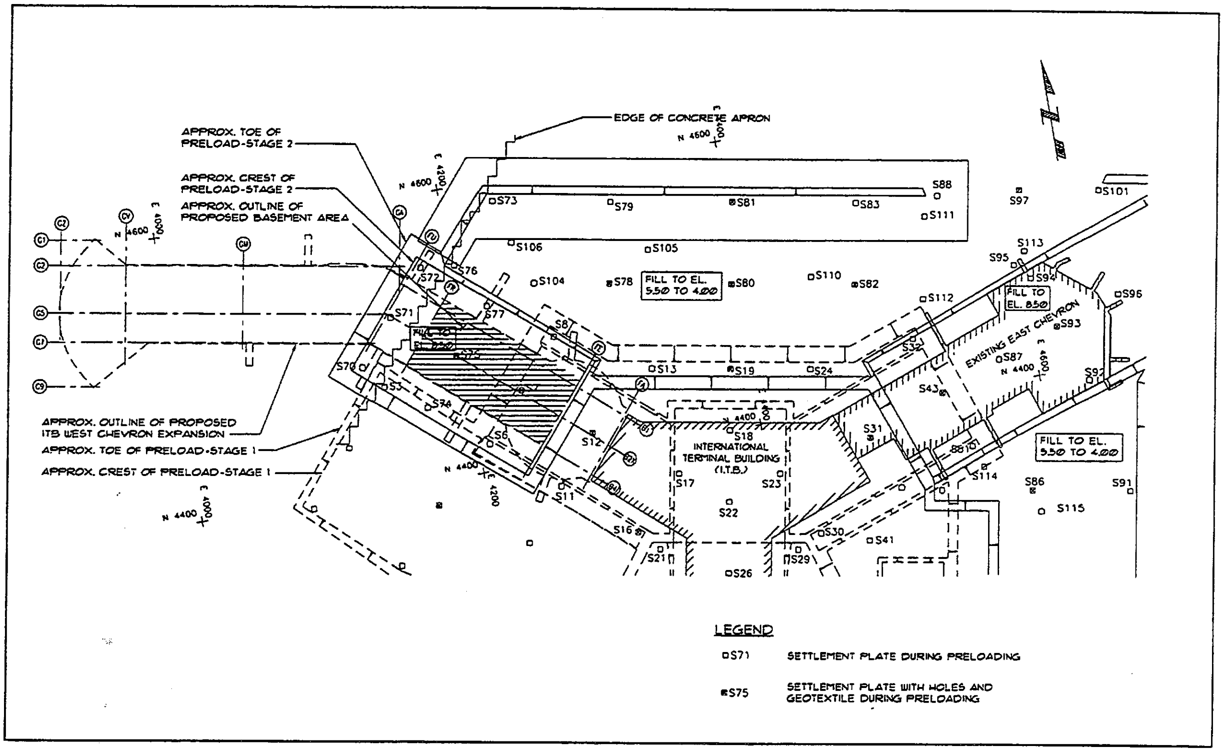
As a consequence of soil liquefaction, lateral soil movements could occur with or followed by post-seismic vertical settlements. Evidence from sites where subsoils liquefied during actual earthquakes in Japan and in the United States indicated that lateral surface soil movements of up to 2.5 m occurred on sloping grounds with gradients of less than 2% (Bartlett and Youd 1995). Empirical methods of estimating the amount of lateral soil displacements due to sand liquefaction under a given seismic event were developed based on the field observations of liquefied sites and correlations with various parameters including: earthquake magnitude, distance to epicentre, subsoil type including grain size, pre- and post-seismic strength and thickness of liquefiable Sand, and ground slope (Byrne 1994). The methods available indicated that, in non-densified areas, lateral soil movements ranging from 250 to 500 mm could occur within the site during a major seismic event. Consolidation type vertical settlements could also occur following sand liquefaction. These vertical settlements would pertain strictly to post-liquefaction consolidation of the sand zone and do not include settlements that could occur due to sand boils or lateral soil spreading. The methods available (Tokimatsu and Seed, 1987) indicated that, in non-densified areas, consolidation type vertical settlements ranging from 100 to 200 mm could occur within the site under a major seismic event. These post-liquefaction settlements could occur immediately after the seismic event over a relatively short duration of about 4 to 6 days. Additional vertical settlements could also occur in the zone 2 upper clayey silt and the zone 3 silty sand layers inter-layered within the sand due to possible deterioration of the strength and stiffness of these soil layers during a major seismic event. The latter vertical settlements could occur over a relatively longer period of time ranging from about 4 to 6 weeks depending on the drainage conditions of the subsoil layers above and below the liquefiable zones.

Site Preparation

Preloading

Previous discussions have highlighted the compressible nature of the subsoils underlying the site when subjected to building and fill loads in excess of natural overburden pressures. It was considered that significant settlements would be generated and that the differential settlements would be beyond structural tolerances. Therefore, a preload treatment prior to construction was required to reduce post-construction settlements to tolerable limits. The function of the preload treatment was to remove a portion of the long-term settlements such that the pattern of differential settlements would be compatible with the structural tolerances of the building.

Fig. 4 Preload layout for the ITB chevron expansions



For illustration purposes, Figure 4 shows the extent of preload treatment carried out since 1992 using sand embankments with a thickness ranging from 4.5 to 12.5 m above original grade.

Based on the interpretation of existing subsurface conditions and review of proposed building and fill permanent loads, preload treatment for the remaining western portion of the west chevron expansion required the placement of a sand embankment fill up to geodetic elevation 7.0 m within 10 m of the building envelope area. The preload duration was expected to be a minimum of 9 months and depended on the interpretation of settlement observations during preloading. The toe of the additional preload for the proposed west chevron expansion building was setback 25 m along the east side from the edge of the existing concrete apron to satisfy current aircraft operations. Consequently, a limited overlap resulted between the previous and new preload fill areas. A preliminary review of the test hole data and the proposed grade increase within the preload overlap area indicated that the building area straddling the limited overlap between the previous and proposed preload fill areas was not expected to result in a more pronounced differential settlements at this location. However, structural expansion joints were considered within the building due to its extended length.

Densification

The design of the proposed ITB chevron expansions was expected to comply with the requirements of earthquake resistant structures as set out in the 1995 NBCC. Since loose to medium dense sand subsoil layers susceptible to liquefaction under the design seismic event were identified, a ground improvement program was recommended to improve the performance of these subsoil layers and allow the proposed building additions to satisfy the requirements of earthquake resistant structures. In order to improve the seismic performance of the zone 3 loose sand layers, densification with vibro-replacement stone columns was carried out. Subsoil densification extended vertically from geodetic elevation EL. -14 m up to the prepared granular working surface which was expected to be at about geodetic elevation 1.45 to 1.75 m. The vibro-densification area extended horizontally a minimum of 7 m beyond the proposed building perimeter column lines to positively cover the influence zone of building perimeter foundation support. After review of the final foundation layout, some adjustment to the horizontal extent of the vibro-densification area was required at the connection with the previously densified ITB area. The densification work was carried out after preloading under a performance specification contract. The performance criteria was based on achieving a minimum Electric Cone Penetration tip resistance to provide a factor of safety of 1.1 against liquefaction. The densification criteria were generally satisfied using 900-mm diameter stone columns located on a triangular equilateral grid pattern with a 3-m centre-to-centre spacing.

Foundation Design

Soil Bearing Capacity

In view of the preload treatment carried out and subsequent to the subsoil densification work, spread footings over a compacted granular base could be utilized to support the proposed building loads. Spread footings were designed using the preliminary allowable soil bearing capacity indicated on Table 2.

Table 2 Preliminary allowable soil bearing capacity

Allowable Footing Bearing Pressure		
D + L	D + L + Q	D + Q
125 kPa	165 kPa	145 kPa

D = Dead Loads L = Live Loads Q = Earthquake Loads

In providing the above allowable soil bearing capacity it was assumed that a compacted granular pad extending a minimum of 600 mm below underside of footing and at least 600 mm beyond the footing perimeter was placed to support the footing. It was also assumed that the footing design loads would be applied vertically. Because inclined design loads and overturning moments were applied on footings, a geotechnical design review was required to confirm the allowable soil bearing capacity actually available for design.

Table 3 Typical load combinations and calculated soil bearing capacity for baggage hall perimeter footings

Loads	V kN	H kN	M KNm	E m	α deg	Q max kPa	Q ult. kPa
D+L	1025	325	800	0.50	11.4	100	744
D+Q Inward	350	200	1025	1.09	12.0	118	652
D+Q Outward	800	550	1875	1.35	21.6	146	417
D+L+Q Inward	800	50	650	0.47	2.1	87	1123
D+L+Q Outward	1250	700	2250	1.25	21.2	180	428

V = Vertical load H = Horizontal load M = Moment
E = Resultant eccentricity α = Resultant inclination
Q max = Max. footing pressure Q ult = Ultimate capacity

For illustration purposes, Tables 3 and 4 provide a summary of the various loading combinations provided by the structural engineer for the baggage hall area and the calculated footing bearing pressures depending on the degree of load eccentricity and inclination.

Table 4 Typical load combinations and calculated soil bearing pressures for baggage hall interior footings

Loads	V kN	H kN	M kNm	E m	α deg	Q max kPa	Q ult. kPa
Service D+L	2750	275	625	0.19	4.7	164	1065
D+Q Inward	1600	375	1325	1.09	12.0	151	787
D+Q outward	1400	75	675	1.35	21.6	112	1150
D+L+Q inward	1850	500	1625	0.47	2.1	176	716
D+L+Q Outward	2650	50	375	1.25	21.2	146	1272

V = Vertical load H = Horizontal load M = Moment
E = Resultant eccentricity α = Resultant inclination
Q max = Max. footing pressure Q ult = Ultimate capacity

The results illustrated on Tables 3 and 4 show that both footing pressure and ultimate soil bearing capacity vary significantly with the degree of resultant inclination and eccentricity for various load combinations. For this example, the ratio of Q max over Q ult indicating the mobilization of soil bearing capacity was found to range from 8 to 43 %. Generally, the highest mobilization of soil bearing capacity corresponded to the combination of dead, live, and earthquake loads.

For illustration purposes, Table 5 outlines the stiffness coefficients calculated for the seismic design of square footings subjected to rocking motions due to a major earthquake for the site after the above outlined site preparation (Dobry et al 1986). The soil shear modulus was the most significant parameter used to derive foundation stiffness coefficients. The soil shear modulus was based on shear wave velocity measurements using Electric Cone Penetration tests.

Table 5 Footing Rocking Stiffness Coefficients

Square Footing Size, B (mm)	Rocking Mode Dynamic Stiffness Coefficient, K_{rx} (MNm/rad)		
	Minimum	Maximum	Median
2000	60	240	120
3000	200	800	400
4000	485	1940	970
5000	950	3800	1900
6000	1650	6600	3300

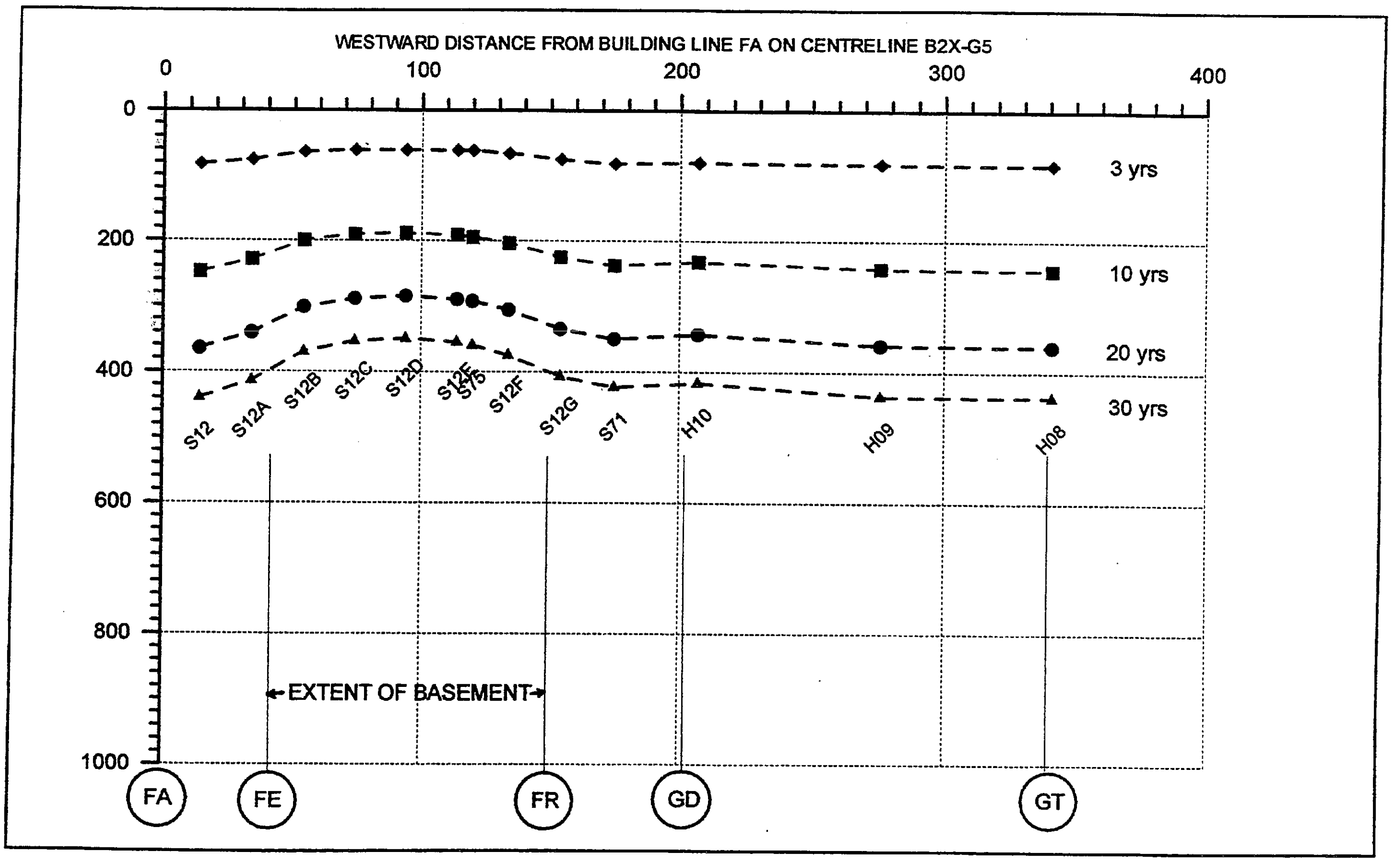
Long-term settlements

The monitoring program set up on the existing ITB structure indicated that the building has settled since its completion in 1995. Therefore, the interface connection between the proposed expansion and the existing structure must accommodate the existing building grade at the time of construction of the expansion. Deeper compressible subsoil strata below the densified subsoil zone will cause significant long-term settlements as indicated by settlement observations of comparable buildings on similar sites over periods in excess of 30 years.

For illustration purposes, Figure 5 shows the estimated patterns of long-term settlements along the proposed west chevron ITB expansion. Theoretical calculated long-term settlements in the order of 300 mm to 500 mm are expected under the permanent fill and building loads considered

A one-dimensional consolidation soil-model was used to estimate the pattern of settlement. The soil-model included both primary and secondary types of settlement and was calibrated using actual field settlement measurements during preloading at Sea Island and from the long-term survey monitoring of existing structures on the Fraser River Delta. Theoretical calculated long-term differential settlements in the order of 1/500 are expected between columns located on typically 10-m bays in both the longitudinal and transverse building directions. The above differential settlement estimates were reviewed with the structural engineers based on the structural design details and available settlement monitoring information.

Fig. 5 Patterns of calculated long-term settlements for the ITB west chevron expansion



Structural Aspects

Introduction

The International Terminal Building at YVR is a large steel moment frame structure that has been designed in four major phases. The building is a significant structure and with its mountain views forms the introduction for most international visitors arriving in British Columbia by air. Three of these phases have been constructed while a fourth phase, the West Chevron, is expected to go to tender within the next year. Figure 1 shows the configuration of the building and shows the existing East Chevron and the future West Chevron phases that are the main subject of this paper.

Table 6 Phases and construction dates for the West Chevron ITB Expansion

Phase	Construction Date	Steel Mass (Tons)	Floor Area (sq.m)	Cost
Original ITB	1994 – 1996	10,000	105,000	$250M
East Concourse	1998 – 1999	760	4,200	$25M
East Chevron	1998 – 2000	3,000	18,900	$65M
West Chevron	2003 – 2005	6,000	29,400	$84M

The building is constructed in a seismic zone on potentially liquefiable soil in the Fraser River Delta where long-term settlements of up to 600mm are expected. Under these circumstances the soil–structure interaction was an important part of the structural design and coordination and mutual understanding of the work of the soils and structural consultant was vital to its successful design.

Cruciform columns and the tree system main structure

The structural form of the International Terminal Building was influenced by both architectural requirements and soil conditions. First, the architecture demanded large open spaces uninterrupted by shear walls or bracing. The extensive network of baggage conveyors that run under the main level of both the original ITB and the West Chevron also required an open plan, without structural elements such as walls or braces that would obstruct operations and make future renovations difficult.

Second, soil conditions precluded the use of bracing systems or shear walls that required tie-downs to resist overturning forces. A structure lighter than concrete was desired to reduce foundation loads. In addition, a structure that could be largely fabricated off-site before the site was available would also offer significant advantage given the tight schedule. To satisfy these criteria, a steel moment frame structure was selected.

For illustration purposes, Figure 7 shows the structural plan of the ITB east and west chevron expansions. To provide stiffness in two directions, a series of frames were spaced at 10 m perpendicular to the axis of the main wings or quadrants of the building and at 20 m in the direction parallel to the axis of these quadrants. The frames intersected, and at these locations column stiffness was desirable in two directions. Cruciform-shaped columns formed from a pair of WT305s and a W610 provided the stiffness required. Each WT had its stem welded to the centre of the web of the W610 to give essentially equal properties in both directions. The cruciform columns flare at the roof into a group of five W310 members, which were chosen to match the flange width of the W610 columns. While one of the W310 members is vertical, the other four are inclined as branches, reducing the span of the roof beams while giving the architectural appearance of a tree. Figure 8 shows a typical cross-section through the main hall. A considerable amount of the steel structure is exposed in the final building. The tree form is most pronounced near the roof where columns are not clad or fireproofed and beams and skylights are exposed.

Seismic design

The building is located in zone 4 for both seismic velocity and acceleration, and governing seismic forces were the lateral loads. Seismic forces were resisted by a series of moment frames in each direction, each with many moment connections. The use of several frames provided for considerable redundancy. In addition to multiple frames in each direction, the ITB contains literally thousands of moment connections that increase the redundancy of the structural system to seismic loads. Detailing was in accordance with S16.1 requirements for a nominally ductile moment frame with an R of 2. The inclined branches at the roof level help to support vertical loads but also transmit lateral loads to the point of inflection of the support columns.

The structural design of the International Terminal Building moment frame takes account of the lessons learnt during the Northridge earthquake of 1994 in which several steel moment frame structures were damaged. In the East and West Chevrons, these include the use of reduced beam sections to force yielding away from the column face flange welds and into the beam.

Fig. 7 Structural plan of the ITB east and west chevron expansions

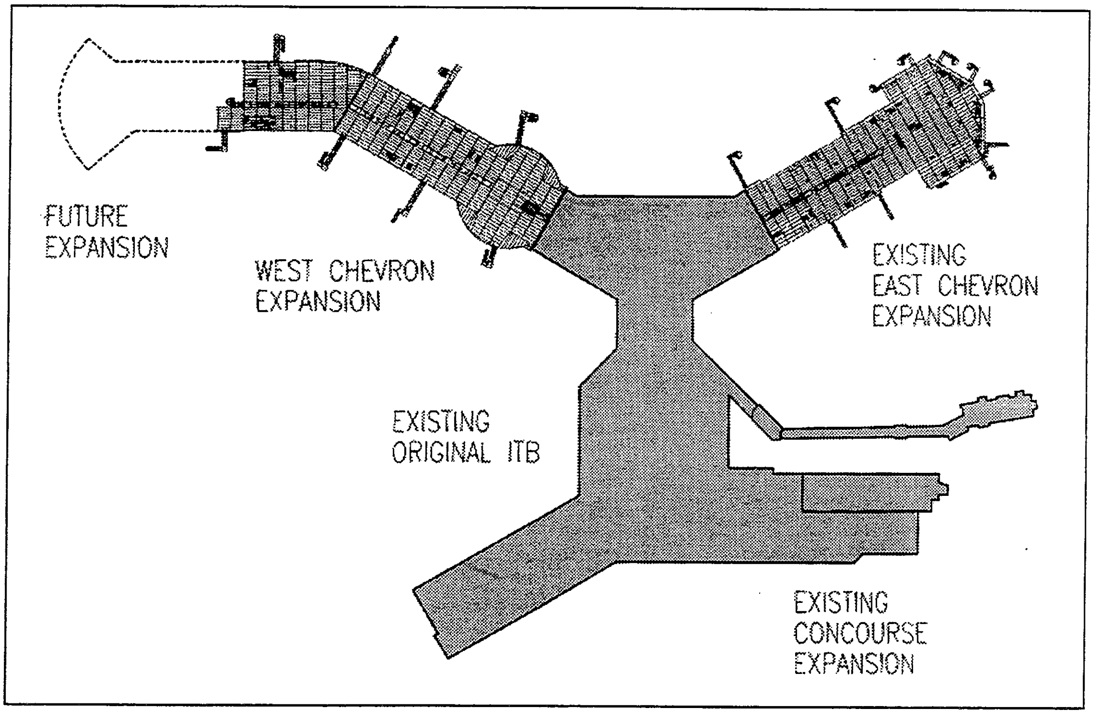
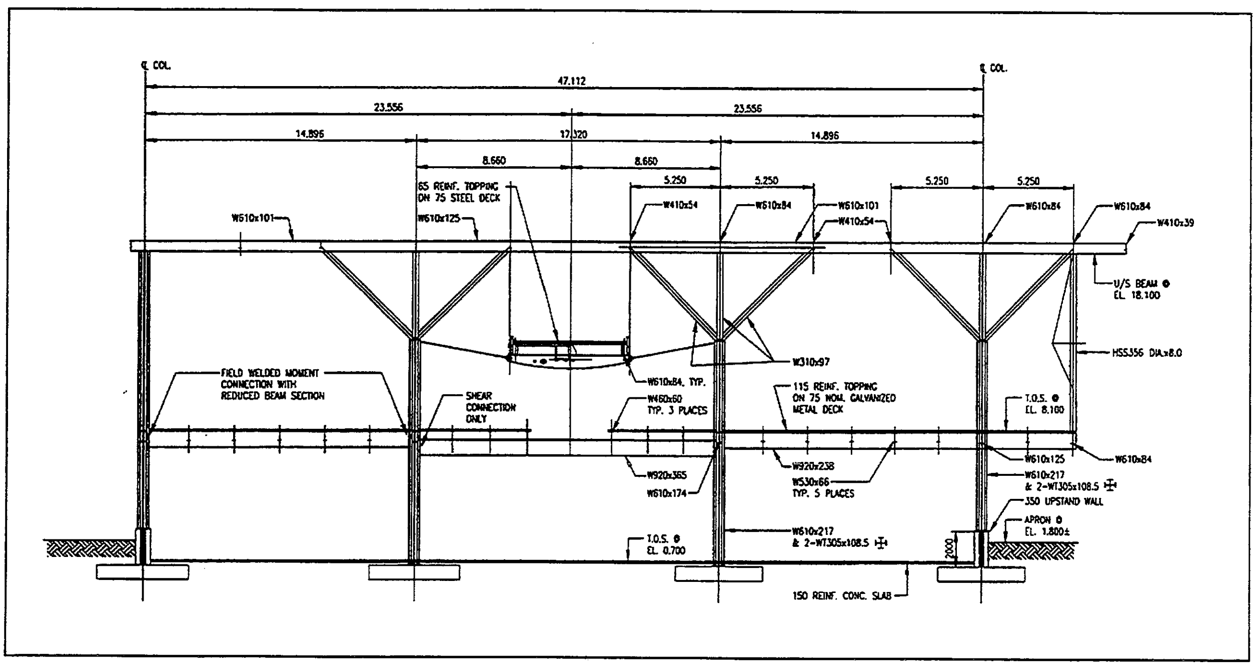


Fig. 8 Section through the baggage hall of the west chevron ITB expansion



Weld problems were highly pronounced in the Northridge earthquake and a comprehensive testing program was undertaken in the International Terminal Building, with all full penetration of welds inspected by non-destructive tests. Weld procedures such as the use of dams, observed to be a problem in Northridge, were avoided. Considerable redundancy in the building and the use of sections only about a third of the size of the 75mm flanges that caused problems in the Northridge earthquake were both positive factors in the seismic resistance of the International Terminal Building. The use of several frames and an R value of 2 results in low ductility and will impose less inelastic rotational demands on the beam-to-column connections.

The structural effects of soil settlement

The significant settlements predicted from the geotechnical analysis are of concern from a structural standpoint as these settlements contribute to stresses in the frame elements. Indeed with a moment frame structure it is possible to overstress the frame from the settlements without any live load being added. To include the effects of long term settlements on the frames the long-term settlement profiles supplied by the geotechnical consultant were applied to the structural models of the frames and were combined with other load cases according to the requirements of the National Building Code of Canada (NBCC).

The 1995 edition of the NBCC does not require that settlement forces be combined with those from seismic design and the code also has lower load factors for live load when combined with settlement loads. From a structural standpoint, it is desirable that the differential settlements are small enough that the load combinations involving settlement do not govern. If load cases containing settlement govern it is possible to get into the situation where members with larger moment capacity must be used, these members are inevitably stiffer than the weaker members and attract more differential settlement forces requiring larger members again – this can result in a spiralling increase and considerably more calculation. Fortunately our studies found that those combinations involving settlement did not govern the seismic design of the frame members on the East Chevron.

What did we as structural engineers really want from the geotechnical engineer

From the structural standpoint the numbers and advice of interest to the structural engineer are:

- **The allowable bearing pressure at the underside of the footing.** This pressure is required under a variety of conditions including dead and live loading, dead and seismic, and dead and live and seismic loading. This is basic information that is needed to size the footings. Our review showed that with the moments that occur on each footing an increase in the allowable bearing pressure often did not have a significant effect on the size of the footing.
- **The expected total settlement and settlement curves.** This is important from the standpoint of determining the effect of the settlement on the moment frame structure. It is also important to note that the settlement curves are of interest from a mechanical standpoint as the settlement has an influence on the direction that water will run on roof slopes and in gravity feed sanitary drains. The mechanical engineer must account for these by having a design that will work both on opening day and after the long-term settlement has occurred.
- **An estimate of any additional differential settlement.** In general this was less important to us than we had thought it would be. The settlement was mostly due to deep-seated long-term settlements and there was little shallow settlement to be added to the settlement curves provided.
- **The rotational spring stiffness for the pad footings.** As the structure is a moment frame the resistance of the column bases to rotation is important. The flexibility of the soil under the foundation means that the column bases lie between being totally fixed and pinned. The geotechnical engineer provided us with rotational spring stiffness values for various sizes of footing. These were included in our frame analysis model for more accurate prediction of expected lateral deflection of the structure and forces in the steel frame elements.
- **The expected water table level for the design of sumps and other below grade structures.** The building is located near the water table and there are some tunnels containing baggage conveyors that will be located beneath the water table. It was necessary to know the expected water table level to design these tunnels such that they did not have positive buoyancy.
- **Soil pressures on walls and tunnels.** There are several retaining and below grade structures that must be designed using pressures provided by the geotechnical consultant.

Communication between structural and geotechnical engineers

The soils information needed by the structural consultant is a function not only of the soil conditions but is also a function of the structure and the anticipated loads. It is not reasonable for us as structural engineers to demand settlement curves if we are not going to provide the geotechnical engineer with our structural configuration and the loads being exerted on the foundations. The communication process started with a preliminary soils report that did not contain a settlement profile and was by intention vague in other areas. As the structural design advanced a series of E-Mails, telephone discussions, faxes, and meetings filled in the gaps with the confidence level building on both sides of the consultancy team.

The importance of footing pressures in moment frame structures.

For columns that do not have moment it is typical for the soils consultant to provide a bearing pressures that are used by the structural consultant to size the footing by dividing the column load by the allowable bearing pressure. The footing is sized the same if the underside of the footing is located at the surface of the soil or at depth. For a moment frame structure this is not the case as the weight of the footing and its overburden have a beneficial effect on the ability of the footing to resist overturning moments. In doing our design we constantly requested bearing pressures at the underside of footings that took account of the weight of the footing and its overburden.

Dead and live loads from a geotechnical standpoint

From our standpoint as structural designers we are faced with several loads to design for – while loads such as seismic and extreme wind loads will occur only occasionally during the life of the structure it is important that the building behave well under dead and live loads. As structural designers we feel we can calculate the dead loads to a reasonable degree of accuracy. The live loads are provided by the building code and for an assembly occupancy building such as the airport the floors are designed for a live load of 4.8 kPa (100 psf). A few quick calculations will show that this is a conservative number. If the average person weighs about 0.75 kN (a-225 lb. football player weighs 1 kN) and if the average person wants to be no closer to fellow travellers than would be permitted in a 1m x 1m square we get a loading of 0.75 kPa, well below the 4.8 kPa design load. From a geotechnical standpoint, using a sustained live load of 4.8 kPa would be unreasonable and as structural engineers we provided the geotechnical engineer an estimate of the sustained live load.

Conclusion

The design of the expansions to the International Terminal Building at Vancouver International Airport challenged both the structural designers and the geotechnical consultants. Using a system of columns that have stiffness in two directions and branches that reduce spans, an economical structural and architectural solution was achieved. The soil-structure interaction was an important part of the design. Favourable results were achieved by extensive communication between the structural and geotechnical engineer.

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