Monitoring Land Subsidence in Sacramento Valley, California, Using GPS

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Abstract: Land subsidence measurement is usually based on a comparison of bench-mark elevations surveyed at different times. These bench marks, established for mapping or the national vertical control network, are not necessarily suitable for measuring land subsidence. Also, many bench marks have been destroyed or are unstable. Conventional releveling of the study area would be costly and would require several years to complete. Differences of as much as 3.9 ft between recent leveling and published bench-mark elevations have been documented at seven locations in the Sacramento Valley. Estimates of land subsidence less than about 0.3 ft are questionable because elevation data are based on leveling and adjustment procedures that occurred over many years. A new vertical control network based on the Global Positioning System (GPS) provides highly accurate vertical control data at relatively low costs, and the survey points can be placed where needed to obtain adequate areal coverage of the area affected by land subsidence.

Introduction

Land subsidence, which is a long-term lowering of the elevation of the land surface, has been observed in the Sacramento Valley and Delta (Fig. 1) since the turn of the century. The causes of land subsidence range from surface erosion by wind to consolidation of subsurface sedimentary materials due to fluid extractions, such as ground-water withdrawal. Various effects of land subsidence in the Sacramento Valley and Delta are shown in Figs. 2–4. Land subsidence has affected both the extent and duration of flooding (Fig. 5) of farmland, homes, and roads, and has reduced the effectiveness of flood channels, levees, and drains.

Irrigation demands in the Sacramento Valley have been met predominantly by surface-water diversions from the Sacramento Valley and its tributaries. As ground-water pumping began to account for more of the irrigation supply in the late 1960s, investigators began to turn their attention to land subsidence in the Sacramento Valley. Attempts were made to document the occurrence of land subsidence by comparing elevations obtained through conventional (differential spirit) leveling over the years. Many of the levees and canals controlling the surface water in the Sacramento Valley traverse great distances and are designed with very gentle gradients (Lofgren and Ireland 1973). Thus, even relatively small reductions (0.5 ft) in land elevation are significant because of the increased potential for serious flooding by overtopping or breakthrough of levees.

Leveling surveys have been made by many agencies in the Sacramento

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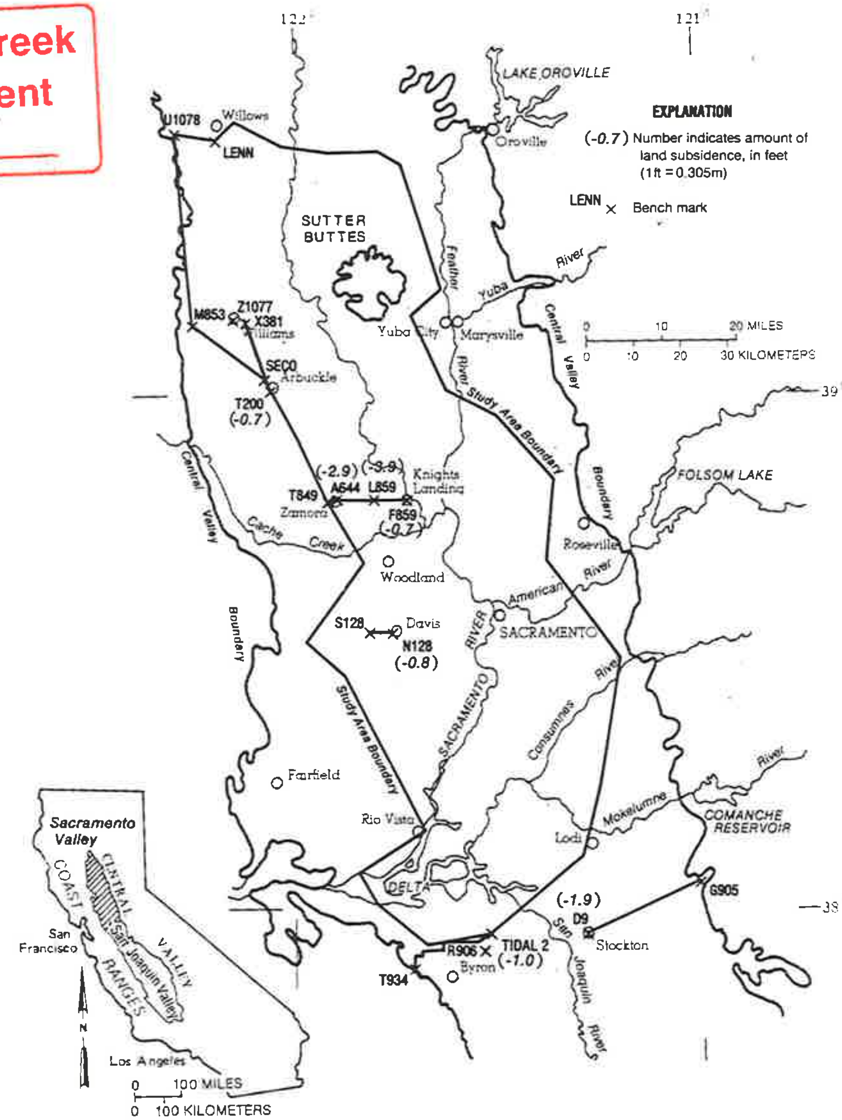

FIG. 1. Survey Area, Selected Bench Marks, and Locations of Documented Land Subsidence in Sacramento Valley

Valley, but the data are inadequate for interpreting the magnitude and rate of land subsidence because most of the surveys have been intended for local mapping control. For land subsidence studies, leveling surveys must be of regional extent and extend beyond the areas of subsidence to relatively stable bench marks on the valley's perimeter. It was found that observed changes in bench-mark elevations provided misleading or incomplete results because

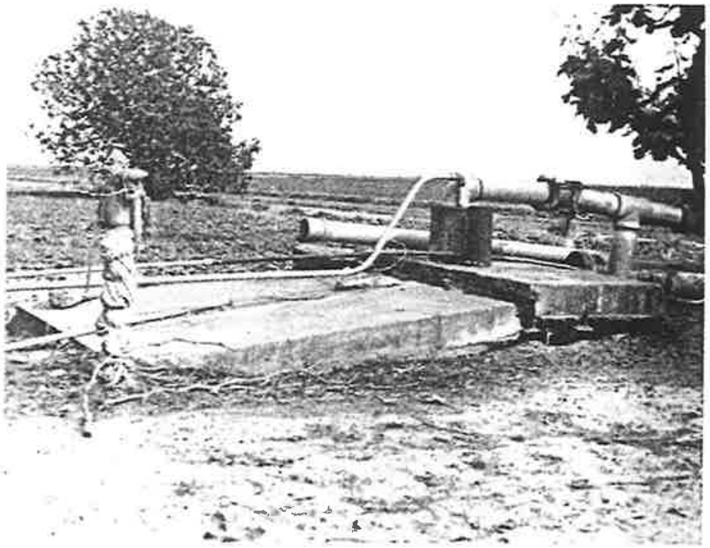

FIG. 2. Submersible Pump with 6-in. Discharge Line near Knights Landing. Well Constructed about 1963. Subsidence Exceeds 1 ft Since 1970. Photographed 1988

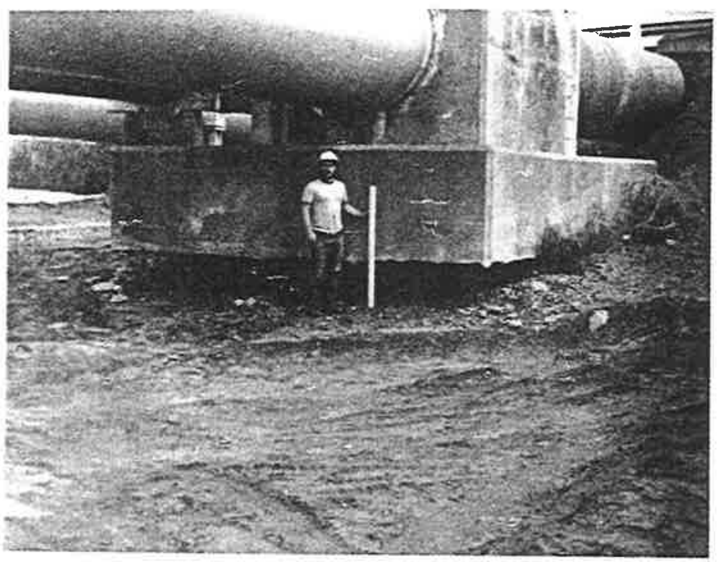

FIG. 3. Mokelumne River Aqueduct near Stockton, Constructed in 1963. Measurement Shows Subsidence of about 2 ft. Replacement Fill along Pipeline Footings Obscures Actual Amount of Subsidence. Photographed 1988

FIG. 4. Anchor for Power-Line Tower Supported by Pilings near Stockton. Constructed about 1930. Subsidence Greater than 5 ft. Photographed 1988

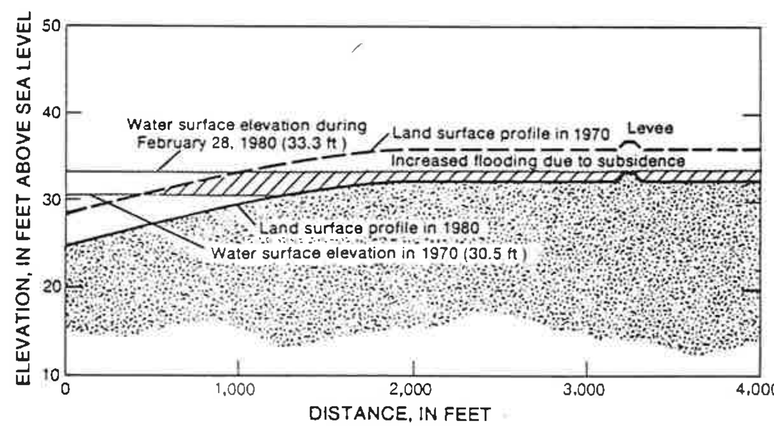

FIG. 5. Effect of Subsidence on Extent of Flooding near Knights Landing

leveling adjustments did not take into account land subsidence, and bench marks were not necessarily located in areas of need. In the search for a new technique to monitor land subsidence, the Global Positioning System (GPS) survey method offered the potential for providing a regional vertical control network with a high degree of accuracy at a reasonable cost, with the freedom to establish a network of bench marks where needed.

History and Causes of Land Subsidence in Sacramento Valley

Lofgren and Ireland (1973) attributed land subsidence in the Sacramento Valley, totaling 0.2–2 ft prior to 1966, to ground-water declines. A study of land subsidence in the Sacramento-San Joaquin River delta and lower Sacramento Valley by Newmarch (1980) indicated subsidence since 1911 of more than 20 ft resulting from depletion of organic soils near the land surface. A later study of delta subsidence by Newmarch (1986) investigated the causes of land subsidence. A major result of that study was an awareness of the limited availability of existing conventional survey data with which to determine the amount and rate of land subsidence.

In the Sacramento Valley, soil compaction due to water-level decline is considered to be the primary subsidence process, with oil-field activities and tectonism possible secondary causes (Williamson et al. 1985). The southern

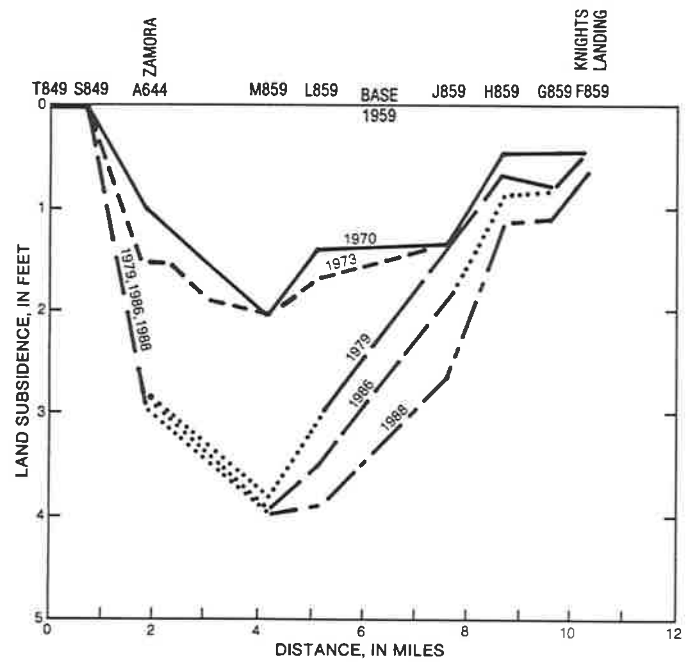

FIG. 6. Amounts of Subsidence Based on Leveling Surveys between 1959 and 1988 on Level Line between Zamora and Knights Landing

part of the Sacramento Valley, extending north from Stockton to Sutter Buttes, contains mostly fluvial deposits from the Sacramento River and its tributaries. These deposits consist predominantly of various strata of gravel, sand, silt, and clay. Page (1986) showed that the aquifer contains 40–60% fine-grained material in the upper 1,000 ft. Page and Bertoldi (1983) noted unusually large compressibility indexes for clay and silt in cores from seven test holes drilled in the Sacramento Valley in 1979 and 1980, indicating the potential for extensive land subsidence in the Sacramento Valley.

Heavy pumping caused newly introduced stresses on aquifers in many places in the Sacramento Valley during the drought in 1976–77. As surface-water supplies diminished during 1976–77, wells were drilled to tap the ground water. Results of leveling done in 1973 and 1979 (one year after the drought ended in spring 1978) show a dramatic drop in bench-mark elevations (Fig. 6). The maximum difference—more than 1.5 ft—occurred in the middle of the level line between Zamora and Knights Landing. Subsidence between 1979 and 1986, however, was estimated to be less than 0.2 ft in this area. The apparent decrease in the rate of subsidence is attributed to the decrease in ground-water pumping after the 1977 drought.

Areas Affected by Land Subsidence

Conventional level lines (Blodgett et al. 1988) with a combined distance of 87 miles were surveyed in the Sacramento Valley (Fig. 1) between 1985 and 1988 to tie GPS surveys to sites of documented ground-water level declines and land subsidence, and to tide gauges (water-level recorders on estuaries) and bench marks set on bedrock. Elevations from bench marks found to be on stable ground were used to define the geoidal separation in the study area. Differences of as much as 3.9 ft between elevations from recent leveling and published bench-mark elevations at seven different locations (Table 1) were discovered.

All locations of documented land subsidence, identified in Table 1, are in areas of significant ground-water pumping except bench marks T934 and TIDAL2, which are in the delta region. Subsidence of peat soils in the delta is attributed to other factors, such as wind transport and oxidation. The lateral extent of land subsidence in the vicinity of Knights Landing (Fig. 6) is

TABLE 1. Comparison of Recent and Older Leveling Data

Bench-mark pairs (1)	Line length (mi) (2)	Elevation difference (ft)		Summary difference (ft) (5)
		Recent leveling (3)	Older leveling (4)	
X381-T200	11	68.40	69.13	0.73
T849-A644	2	67.01	64.07	2.94
T849-L859	5	73.85	69.91	3.94
T849-F859	11	75.17	74.46	0.71
S128-N128	3	26.01	25.20	0.81
T934-TIDAL 2	14	207.35	206.38	0.97
G905-D9	18	132.89	130.97	1.92

Note: Location of bench marks shown in Fig. 1.

more than 10 miles. Surveys to determine the areal extent of subsidence at various other sites will be determined during future GPS surveys.

Land Subsidence Measurement before Use of GPS

Apparent Rate of Subsidence Based on Leveling Data

Estimates of the rate of subsidence are usually based on a comparison of historic and present elevations of local landmark features. For example, the average rate of subsidence of the power-line-tower anchor shown in Fig. 4 has been estimated from the amount of settlement (5.3 ft) since construction of the anchor in 1930. The estimated average rate of subsidence for this site for 1930–88 would be about 0.1 ft/yr. The reliability of such estimates, however, is questionable because construction of the anchor footing and soil conditions at these sites are uncertain.

Estimates of land subsidence also can be obtained from a comparison of historic bench-mark elevations. The relative stability of bench marks can be identified by comparing historic adjusted elevations; those indicating little change are considered to be more stable. Differences in elevation less than about 0.3 ft were not considered reliable indicators of land subsidence in the Sacramento Valley. Because the data are based on measurements collected over many months, during which time subsidence is occurring, adjustment procedures tend to obscure the magnitude of subsidence; therefore, estimates using these data may not be accurate.

Bench-mark elevations published by the National Geodetic Survey (NGS) may reflect changes from both land subsidence and adjustment procedures (Table 2). Referring to Table 2, the published elevation of bench mark T934 near Byron increased 0.085 ft between 1960 and 1975. This bench mark is situated on a stable sandstone outcrop of the Upper Cretaceous and Paleocene Moreno Formation (Pampeyan 1964; Brabb et al. 1971), so no actual change in the elevation of this bench mark would be expected. Therefore, changes in the elevation of this bench mark during this period are probably related to adjustments needed to balance the level networks in this area rather than land subsidence or uplift at this site.

In another example, successive elevations of bench marks S128 and N128 suggest land subsidence ranging from 0.4–1.2 ft between 1932 and 1969 (Table 2). In actuality, some of the change in elevation of these bench marks is attributed to adjustments of the level network. Assuming all of the change in elevation is related to land subsidence, the apparent rate of subsidence of bench mark N128 (1.2 ft over 37 years) would be about 0.03 ft/yr. In these

TABLE 2. History of Selected National Geodetic Survey Bench-Mark Elevation Adjustments

Bench mark (1)	NGS line number (2)	Date of Adjustment and Elevation (National Geodetic Vertical Datum of 1929) (ft) 1932 (3)	1935–42 (4)	1949 (5)	1959 (6)	1960 (7)	1963–64 (8)	1966–67 (9)	1969 (10)	1975 (11)	1986 (12)
T934	105	—	—	—	221.355	221.355	—	—	—	221.440	—
S128	106	63.635	63.589	—	—	—	63.340	63.232	63.232	—	—
N128	106	38.435	38.340	—	—	—	37.556	37.274	37.218	—	—
L859	103	—	—	46.48	—	—	—	—	—	—	42.96

Note: Location of bench marks shown in Fig. 1.

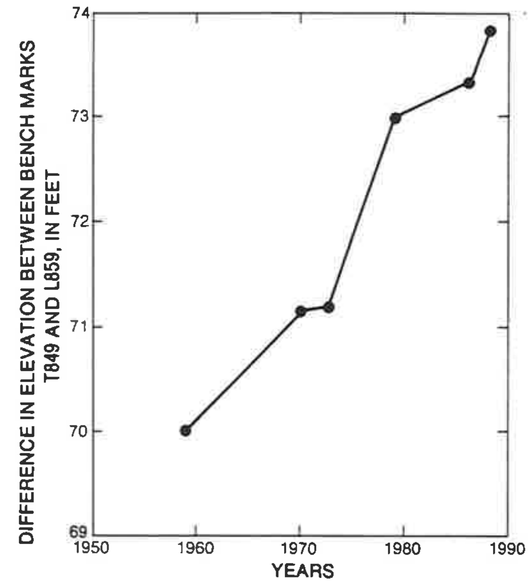

FIG. 7. Rate of Subsidence for 1959–88 at Bench Mark L859 near Zamora

example estimates of apparent subsidence, it was assumed that the rate of subsidence was constant during the period of analysis.

Land subsidence generally does not occur at a uniform rate during a period of time; therefore, the previous examples may provide misleading information on both the magnitude and rate of subsidence. Referring to bench mark L859 (Table 2), located in an area of known subsidence, the apparent rate of subsidence was 0.09 ft/yr between 1949 and 1986. A comparison of differences in elevation for bench marks T849, located on stable terrain, and L859, located on unstable ground (Fig. 7), indicates that the rate of subsidence at this site is not uniform with time, but depends on factors such as the rate of ground-water withdrawals.

Effect of Bench-Mark Construction and Placement

Bench marks are generally brass tablets placed on concrete or steel posts located on levees and highway or railroad right-of-way properties. Other common locations of bench marks are on the concrete headwall, abutment, or footing of a bridge, on culvert and irrigation gate headwalls, on building and power-transmission-tower foundations, and in sidewalks. Some bench marks are placed on steel rods driven into the ground to refusal, sometimes over 100 ft deep. In some cases, the brass tablet is cemented into a bedrock outcrop or boulder. Unfortunately, many bench marks are subject to damage or removal by roadway and utilities construction or farming operations.

Elevation data for bench marks constructed on nonrepresentative foundation material can result in inaccurate estimates of land subsidence. The stability of a bench mark is related to its construction, type of foundation material, and location. For example, NGS bench mark R906, constructed in 1957 in the delta near Byron, is supported on steel rods driven to refusal at

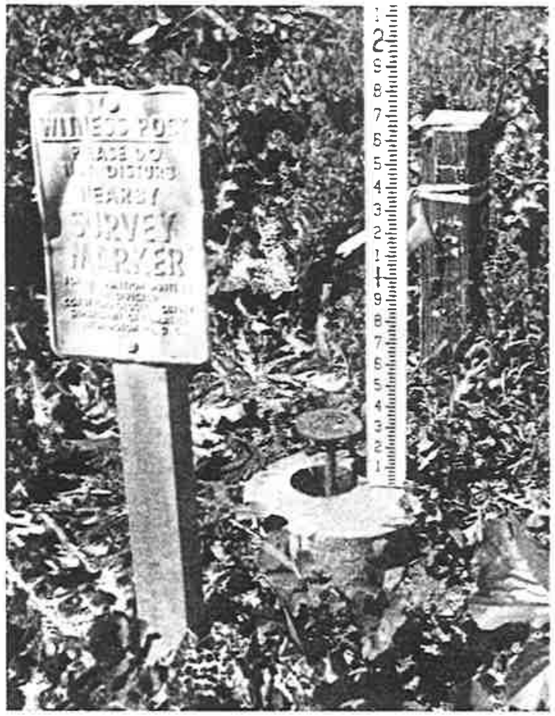

FIG. 8. Bench Mark R906 near Byron. Photographed 1988

61 ft (Fig. 8). Since 1957, the land surrounding this bench mark and the concrete post have subsided about 0.3 ft more than the brass tablet and rod. Bench marks on structures supported by pilings, however, generally are not affected as much by land subsidence or erosion (Fig. 4). In other cases, bench marks placed on concrete posts located on levees reflect subsidence of the levee plus a surcharge reflecting compression of the soils beneath the levees. For example, bench mark TIDAL2 (Fig. 1), which is located on a railroad fill, subsided about 0.5 ft between 1967 and 1987. This subsidence is attributed to consolidation of the underlying peat and other surficial material plus compaction of the railroad embankment.

Effect of Errors in Leveling Data

Basing estimates of land subsidence on a comparison of present and historical bench-mark elevations can be misleading. For example, the difference between recent GPS surveys and published historic elevations for bench marks on the route SECO-M853-U1078 (Fig. 1) is 2.01 ft. However, using the route SECO-Z1077-LENN-U1078, the difference is 0.01 ft. The reason for the differences in bench-mark elevations derived from these two level lines is unknown. As shown by this example, some estimates of land subsidence based on elevations of bench marks derived from conventional leveling may be unreliable. However, because the occurrence and magnitude of level line closures are inconsistent and application of adjustment procedures uncertain,

it is difficult to determine which elevation data are unsuitable. Therefore, all data are used with caution in the analysis of land subsidence.

Three factors cause inaccuracies (referred to as noise) in estimates of land subsidence: (1) Problems relating different level survey adjustments—errors related to adjustment of level closures; (2) errors caused by leveling equipment and procedure; and (3) effects of bench-mark location and construction.

Studies of misclosure of level circuits caused by refraction (Holdahl 1983) reveal the necessity for applying corrections for this type of error. This error is caused by vertical temperature gradients near the ground that cause bending of the line of sight and can cause a significant error in leveling results where sight distances are long on sloping ground. Much of the U.S. Vertical Control Network prior to 1964 was affected by this error (Holdahl 1983), but after 1964 average sight distances were reduced to about 150 feet, which tended to reduce this type of error. Changes in instrumentation and leveling procedures since 1964 have caused problems in land subsidence studies because old and new leveling surveys over the same route appear to show relative vertical movement if corrections for refraction are not applied. Consequently, without refraction corrections to old levelings over the same route, it may not be possible to distinguish between refraction errors, other survey errors (such as instrument errors), and land subsidence.

Errors in field leveling surveys are related to the type of equipment used and the surveying procedure. For second-order class I leveling, errors related to field procedures should not exceed $0.0250\sqrt{L}$ feet (where L is length of level line, in miles) in the closure of a loop ("Classification" 1974). For a line 40 miles long, which is the average width of the Sacramento Valley, an acceptable closure in field surveys would be $0.0250\sqrt{40} = 0.16$ ft.

Problems related to leveling adjustment procedures are more common in areas of subsidence in which adjustments are based on a local level network rather than a regional network. A further problem occurs when adjustments for different level lines in an area are merged to form a common network even though the surveying was completed in different years. In addition, surveys tied to various bench marks may be affected by varying amounts of land subsidence. In effect, the adjusted elevations may be less accurate than those from the original field surveys.

The combined effects of errors in the closure adjustment procedure and field leveling errors for second-order class I surveys result in a noise level that ranges from 0.1–0.3 ft, depending on the line length, as shown in Fig. 9. The noise level may be calculated using the equation

$$\text{Noise level} = [(\text{leveling standard})^2 + (\text{adjustment procedure error})^2]^{1/2} \quad \ldots . \ (1)$$

where leveling standards are for second-order class I surveys, and adjustment procedure error is based on field data for 17 survey lines in the Sacramento Valley. The relation of vertical control inaccuracy, or noise, as a function of survey line length was determined. An analysis of level adjustment closure errors for the 17 survey lines resulted in an adjustment procedure error of 0.114 ft. The curve in Fig. 9 does not include the possible effects of bench-mark instability related to location or construction. Therefore, estimates of land subsidence based on conventional leveling procedures less than about 0.3 ft may not be considered reliable.

Because a comparison of existing bench-mark elevations often results in questionable estimates of land subsidence, a different method of document-

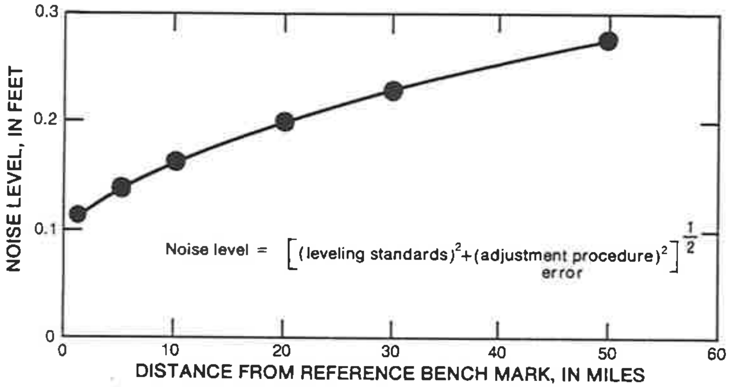

FIG. 9. Estimates of Conventional Leveling Accuracy

ing present and future land-surface elevations was needed. A new method of establishing vertical control data, based on GPS surveys, has been utilized in the Sacramento Valley.

GPS Survey Technique

GPS is a U.S. Department of Defense satellite-based navigation system designed to provide continuous worldwide positioning capability. Six production (block II) and six prototype satellites are currently operational. A constellation of 21 satellites is planned for a fully operational system in 1992. User field equipment comprises a small antenna and a receiver-processor unit, which records data on cassette tape. When the GPS system is used for surveying, precise relative positions of two or more points are determined from simultaneous satellite tracking data acquired at each point. Various algorithms are used to process the data, but the initial results are always given in a three-dimensional rectangular coordinate system. A transformation converts the rectangular coordinates into geodetic latitude, longitude, and height, relative to an arbitrary reference ellipsoid. Because elevations determined using conventional leveling are relative to the geoid—the equipotential surface coinciding with sea level—they differ from GPS ellipsoidal heights by an amount equal to the height of the geoid above the ellipsoid (Fig. 10). This quantity, the geoidal separation, ranges from about 59–131 ft in the conterminous United States relative to North American Datum 1983 (NAD83) (Collins 1988). The slope of the geoid has a root mean square of about 25 parts per million (ppm) in level areas (W. M. Kaula, National Geodetic Survey, written communication, 1985) and can exceed 75 ppm in mountainous regions.

Ellipsoidal heights and conventionally leveled elevations also differ in that ellipsoidal heights are measured normal to the ellipsoid, whereas conventional elevations are measured normal to the geoid (Fig. 10). The difference in the directions of the normals, the deflection of the vertical, however, is typically very small, usually seconds of arc. Changes in elevation, as caused by subsidence, for example, are equivalent to changes in ellipsoidal height obtained during successive GPS surveys. Therefore, elevations of bench marks

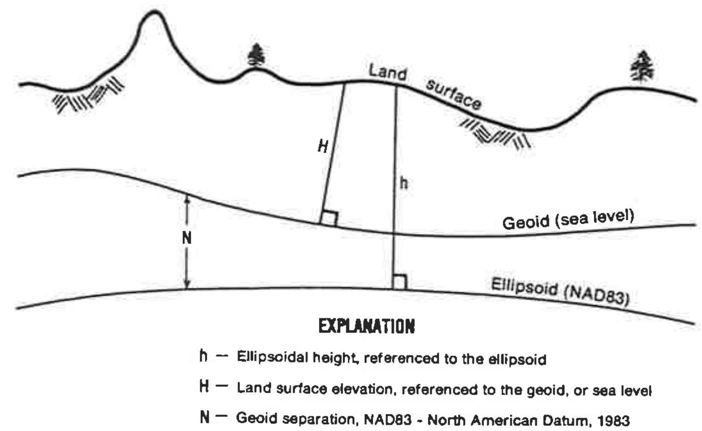

FIG. 10. Relation of Ellipsoidal, Geoidal, and Land-Surface Heights

referenced to sea level cannot be redefined on the basis of GPS surveys without an accurate knowledge of the geoidal separation applicable to the study area. Changes in elevation can be monitored by repeat GPS surveys, which measure changes in ellipsoidal height with an accuracy limited only by the inherent errors in the GPS technique, presently about 1.5 ppm of the distance between points (Bock et al. 1984).

Reconnaissance for GPS Surveys

One advantage of the GPS technique is the flexibility it allows the user in designing the survey network. Line of sight between the end points of the survey line is not required, and very high accuracy can be maintained over lines many miles long. Also, the geometrical constraints inherent in conventional triangulation and traverse networks do not apply to GPS surveys. Consequently, it is relatively easy to construct a network incorporating only the points of interest. GPS surveys do have one significant limitation in that signals transmitted by the satellites can be blocked by obstructions between the satellite and receiver antenna. When there are obstructions higher than 20° above the horizon at the bench marks, GPS sites are offset from bench marks to obtain adequate satellite visibility.

The following criteria were used in selecting sites for GPS surveys.

- History of land subsidence or flood-related problems in an area and, conversely, a history of geologic and tectonic stability.
- For the primary network, published bench-mark elevations that show little change with time.
- For the secondary network, published bench-mark elevations that show large changes with time.
- Potential for urban development in an area.
- Condition of existing bench marks.
- Distance between bench marks.
- Areal distribution of bench marks.
- Accessibility of site for vehicles and receiver setup.

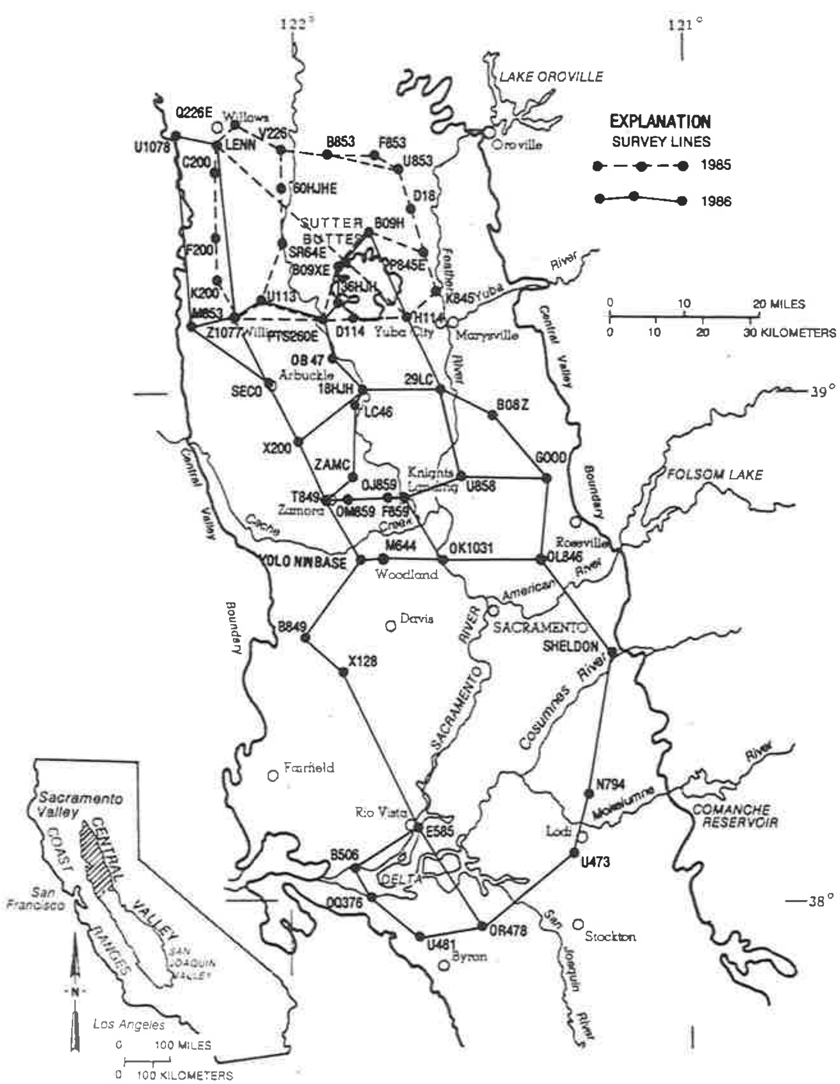

FIG. 11. Networks for 1985 and 1986 GPS Surveys in Sacramento Valley

Description of GPS Survey in 1985

The 1985 GPS survey included 21 bench marks in an area of 1,400 sq mi (Fig. 11) and was made by Aero Service using Macrometer II Dual-band Interferometric Surveyors (the use of brand or trade names in this report is for identification purposes only and does not imply endorsement by the U.S. Geological Survey) ("Report" 1983). The data were reduced using software based on satellite ephemerides produced by Aero Service. A simultaneous least-squares adjustment of the vector data was made to obtain the final station coordinates. Survey results were well within specified requirements. Details of the survey are given by Ladd (1986a, b). A summary of 1985 GPS field operations and data processing follows.

- Equipment: Two Macrometer II Dual-Band Interferometric surveyors
- Survey area (NAD27): Latitude, 39°08′ to 39°31′; longitude, 121°38′ to 122°11′; elevation, 48.7 to 115.9 ft; area, approximately 1,400 sq mi.
- Survey period: July 13–28.
- 21 bench marks: eight bench-mark offsets required.
- Observation spans: Primary network, four lines, 3.5-hour session; secondary network, 26 lines, 1-hour session.
- Consistency of GPS network, based on repeat surveys: Test number 1; baseline length, 34,513.453 ft; delta latitude, 0.00017 in.; delta longitude, 0.00025 in.; delta ellipsoidal height, 0.036 ft; accuracy, 1 ppm horizontal vector, 1 ppm vertical vector. Test number 2; baseline length, 121,383.577 ft; delta latitude, 0.00028 in; delta longitude, 0.00100 in.; delta ellipsoidal height, 0.174 ft; accuracy, 1 ppm horizontal vector, 1.6 ppm vertical vector.
- Survey closure statistics: Average line length, 34,513 ft; average (n = 30) difference between measured and adjusted vector lengths in terms of vector lengths, 1.7 ppm.

The geoidal separation at each survey point was computed by differentiating the published elevation and the computed ellipsoidal height (based on the NAD27 datum and approximate ellipsoidal height at bench mark LENN). At the time of the survey, LENN was considered to be on stable ground. The point values were contoured to construct a geoid map of the project area relative to the assumed ellipsoidal height at LENN. The contour map was not used in subsequent subsidence analyses, however, because the accuracy of many calculated geoidal separation values was uncertain, such as for LENN and U853.

Description of GPS Survey in 1986

The 1986 GPS survey included 38 bench marks in an area of 5,300 sq mi (Fig. 11), and was made by the National Mapping Division of the U.S. Geological Survey based on specifications for GPS relative positioning techniques ("Geometric" 1988). The objective of this survey was to extend the 1985 GPS network to the southern part of the Sacramento Valley. The survey was designed to:

1. Provide sufficient reobservations of stable bench marks in the 1985 survey to confirm GPS repeatability and to relate the 1985 and 1986 networks.
2. Produce elevations for network points using GPS ellipsoidal heights corrected for geoidal separation. These elevations would be compared with historical conventional leveling data to identify subsidence areas.
3. Create a network of points with precise GPS ellipsoidal heights so that future subsidence monitoring could be accomplished by comparing repeat GPS surveys.
4. Provide precise horizontal coordinates of bench marks in the network for NAD27 and NAD83 datums.

Results of the 1986 GPS survey are summarized as follows.

- Survey area (NAD27 and 83): Latitude, 37°54′ to 39°31′; longitude, 121°12′ to 122°19′; elevation, 5.4–305.9 ft; area, approximately 5,300 sq mi.
- Survey period: August 5–November 1.
- 38 bench marks: nine bench-mark offsets required.
- Observation spans: Primary network, four lines, 5-hour session; secondary network, 43 lines, 2-hour session.
- Survey closure statistics: Average line length, 52,793 ft; Average (n = 47) difference between measured and adjusted vector lengths in terms of vector lengths, 1.6 ppm.

Satellite SV9 was inoperable for part of the project, resulting in suboptimal definition of the satellite path geometry. The data were reduced using the PHASAR software package developed at NGS by Goad (1985). The broadcast satellite ephemerides were used in the computations. The vector network was adjusted and transformed from the satellite (WGS72) system to the local datum using the NASSTI program (Vincenty 1982). Several different adjustments were made to satisfy different objectives, as follows.

The first adjustment was a minimal-constraint solution, holding station LENN fixed at the published NAD83 position. The fixed ellipsoidal height of LENN was computed by correcting the elevation of LENN for the geoidal separation. This adjustment was used to assess internal consistency of the GPS network and to compare results with the 1985 GPS survey results.

The second adjustment was made with stable bench marks around the periphery of the project area fixed at their ellipsoidal heights as determined from published elevations corrected for geoidal separation. The horizontal coordinates of two points were constrained to NAD83 values. The GPS network was allowed to rotate about the X, Y, and Z axes to bring it into coincidence with the local system. This adjustment resulted in a set of ellipsoidal heights for the network points in a system defined by the published elevations of the fixed points and the geoidal separation model. The adjusted ellipsoidal heights were then corrected for geoidal separation to obtain elevations for comparison with historical level data. Elevation data obtained using this procedure were used primarily to identify outlier bench-mark elevations, to which levels were run using conventional survey procedures. In general, this procedure indicated that some very old level lines (levels run about 1908) were tied to a different datum.

Other adjustments were made to produce NAD27 and NAD83 coordinates for the 1986 survey and for the combined 1985 and 1986 surveys. Five NGS horizontal control points were held fixed in these adjustments. Agreement between the GPS survey and the NAD83 coordinates was at the 3 ppm level, which is the accuracy claimed for the NAD83 points. Agreement with the NAD27 coordinates was not nearly as good, with differences in excess of 3 ft at some stations.

Comparison of 1985 and 1986 GPS Surveys

Differences between the 1985 and 1986 GPS surveys, during which a total of 59 vertical control points and 77 connecting control lines were established, are given in Table 3. In areas where land subsidence is not a factor, a comparison of sequential GPS surveys in 1985 and 1986 was made for nine lines, ranging in length from 5.3–23.0 mi. This comparison indicated an av~~age difference of 0.09 ft between the repeat surveys, well within the

TABLE 3. Comparison of 1985 and 1986 GPS Survey Results (NAD 1927)

Bench-mark pairs (1)	Line length (mi) (2)	Elevation Difference (ft)			Summary difference (ft) (6)	Differences (Parts Per Million)	
		Published (3)	1985 (4)	1986 (5)		Distance (7)	Ellipsoidal height (8)
P845E-D18	6.54	16.556	17.720, 17.757	—	0.037	—	—
LENN-Z1077	23.0	36.902	40.130, 40.304	—	0.174	1.1	2.6
Z1077-U113	5.30	25.933	24.965	24.854	0.111	—	4.0
U113-PTS260E	8.67	6.00	4.461	4.479	0.018	—	0.2
PTS260E-B09XE	7.68	22.788	24.336	24.155	0.181	1.2	2.7
B09XE-B09H	6.74	0.494	1.886	1.883	0.003	0.1	0.1
B09H-H114	13.0	21.989	23.118	22.951	0.167	1.0	2.4
PTS260E-H114	10.2	1.293	3.104	3.087	0.017	6.6[a]	0.2
Average					0.09	2.0	1.7

[a]Field log indicates that tripod may have shifted during observations.
Note: Location of bench marks shown in Fig. 11. Suffix E indicates offset location from bench mark.

accuracy needed to detect land subsidence. Differences in data for both surveys, which were adjusted to the NAD83 coordinate system, were compatible with the estimated accuracy of 1–2 ppm for each survey. The results are particularly encouraging if one considers that they were obtained with different hardware, software, and satellite geometry at different periods of time.

Comparison of Recent Conventional Leveling and GPS Survey Data

A comparison of concurrent leveling done by the U.S. Geological Survey using conventional (spirit) leveling and GPS surveys at six sites resulted in an average difference of 0.08 ft between the methods (Table 4). The GPS elevations were computed using geoidal separation data from the NGS collocation program. The data in Table 4 indicate a linear relation between elevation differences for the two procedures and line length (Fig. 12).

Geoidal Separation Data in Study Area

The analysis of geoidal separation data was made using two different data

TABLE 4. Comparison of Conventional Leveling and GPS Data

Bench-mark pairs (1)	Line length (mi) (2)	Elevation Difference (ft)		Summary difference (ft) (5)
		USGS leveling (third order) (3)	GPS (4)	
T849-F859	10.4	74.96	74.84	0.12
T849-OM859	4.1	74.14	74.11	0.03
T849-OJ859	7.8	76.99	76.89	0.10
U481-OR478	8.1	54.13	54.04	0.09
F859-OM859	6.3	0.82	0.74	0.08
OM859-OJ859	3.7	2.85	2.79	0.06
Average				0.08

Note: Location of bench marks shown in Fig. 11. Prefix O indicates offset location from bench mark.

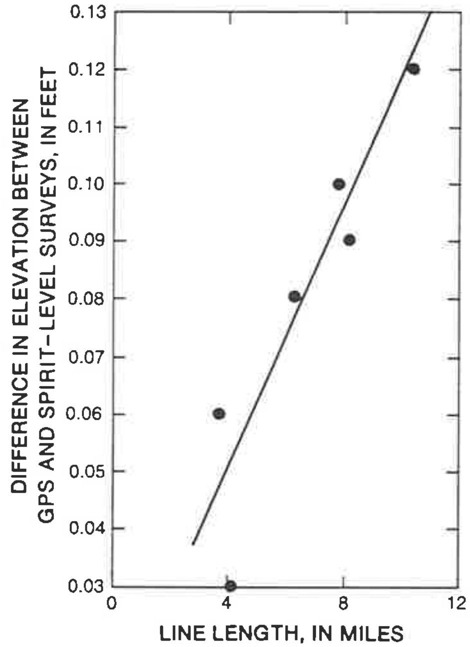

FIG. 12. Relation of Line Length to Difference between Elevations Obtained by GPS and Spirit-Leveling Surveys

sets. The first set was derived by NGS from gravity and deflection data using least-squares collocation techniques. The accuracy of separations computed in this manner is primarily a function of the density and distribution of data points near the project area. Given sufficient data, this technique will reflect small irregularities in the geoid. On the basis of the results, it is estimated that relative geoidal separations between adjacent bench marks in the same 30-minute block are accurate to about 0.10 ft.

The second set of geoidal separation data was produced by NGS using a RAPP 360 × 360 geopotential model. In general, geopotential models do not depict short-term variations in the geoid; therefore, one would expect the collocation data to provide the best answers (Schwartz et al. 1987). Better agreement with known stable elevation data was obtained when using the collocation data. As a result, these data were used in the final analysis.

Summary

Estimates of land subsidence have, until now, been based on a comparison of bench-mark elevations surveyed over a span of years. Existing vertical control has been inadequate for monitoring subsidence in the Sacramento Valley. GPS survey data facilitate subsidence studies because vertical control data obtained from sequential surveys provide point-in-time vertical control data that can be compared with subsequent surveys to indicate rates of subsidence accurately.

A new vertical control network in the Sacramento Valley has been developed. Because the cost of updating and densifying the vertical control network by conventional leveling was prohibitive, a decision was made to test the GPS survey technique. This technique allows the use of survey points placed where needed to obtain adequate areal coverage of the region affected by land subsidence.

GPS surveys were made in 1985 and 1986, during which a total of 59 vertical control points and 77 connecting control lines were established. A primary network of stable bench marks and a secondary network were established to monitor areas of known or suspected subsidence. Conventional levels with a combined distance of 87 miles were surveyed to tie GPS surveys to sites where bench marks were set in bedrock and where groundwater levels and land consolidation were being measured. Elevations from bench marks found to be on stable ground were used to define geoidal separation in the study area.

An average difference of 0.08 ft between leveling by conventional and by GPS methods at six sites was obtained. In areas where land subsidence is not a factor, a comparison of sequential GPS surveys in 1985 and 1986 was made for nine lines, ranging in length from 5.3–23.0 mi. This comparison indicated an average difference of 0.09 ft between the repeat surveys, well within the accuracy needed to detect land subsidence. These surveys demonstrate that GPS is a cost-effective means of detecting and monitoring land subsidence, and that the control network established in 1985 and 1986 can be expanded.

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Appendix I. References

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Appendix II. Conversion Factors

To convert	To	Multiply by
in.	mm	25.4
ft	m	0.305
ft/mi	m/km	0.189
ft/yr	m/annum	0.305
mi	km	1.61
sq mi	km^2	2.59

GIS: Useful Tool or Expensive Toy?[a]

By Roger B. Arend[1]

Abstract: The relatively new technology known as geographic information system (GIS) has many possible uses for engineers who have projects covering large geographic areas. The many categories of data that can be included in a GIS provide the user with valuable resource data that can be combined with or subtracted from other categories to create entirely new categories. GISs come in two basic forms—raster or vector—and each has its advantages and disadvantages. Raster systems assign attributes to specific ground locations, and the resultant output is in the form of typewriter symbols, while vector systems assign attributes to polygons and have outputs that are visually more attractive. GIS technology is widely used by planners, and use by engineers is increasing. Advances in marrying the GIS software with better graphics reproduction software and increased exposure to GIS have brought about increased use by engineers.

Introduction

Computers and computer software have provided the means to achieve accuracies in mapping that once were beyond our wildest dreams. Why is it, then, that computerized geographic information systems (GIS) should have applications for engineers? What more than a good computer-aided drafting (CAD) system does an engineer need?

GISs permit the user to filter through myriad data to discover the exact set of attributes that will answer a question. Thus, any system that permits managing, storing, analyzing, and displaying goegraphic information is a GIS. A single thematic map is a GIS. A series of paper, or analog, maps with attendant transparent overlays forms a GIS. For the purposes of this paper, however, GIS will refer to the rapidly growing field of computer management of graphic map data bases, sometimes called by other names, such as land information systems, geo-based information systems, multipurpose cadastres, and geo-data systems.

The term *system* as used in GIS is a misnomer, since GIS is really a technology. The computer hardware and any particular brand of GIS software (the system) are just a small part of the group of geographic data-base software that make up the technology (Parker 1988). GIS software currently has the ability to rapidly and efficiently acquire, store, analyze, manipulate, alter, combine, and display geographically referenced data throught the use of computer hardware and software systems.

Characteristics of GIS

In order to explain GIS, it is necessary to discuss the characteristics that make GIS unique and to give some examples of more commonly understood

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