Economics of Low-Rise Steel-Framed Structures

JOHN L. RUDDY

The structural engineers' input at the conceptual state of a project is his most important contribution. The purpose of this early effort is to determine the materials of construction, and to establish the system which is to use these materials. Inadvertently, these early decisions establish the lower boundary of the economies that can be realized as design progresses.

Options available to the engineer are numerous. In addition to the multiple framing options, within the constraints of each option, there are a multitude of permutations of elements which further magnify the options. Confidence that a true optimum solution has been realized seldom exists. The magnitude of the problem certainly does not diminish its importance. Our purpose here is to investigate the structural economic considerations of a one-story framing system in the interest of predicting the optimum bay dimensions.

The objective of the investigation is to realize least cost. Least cost is *not* synonymous with the least weight of the structural steel elements employed. In fact, a least weight approach does not consider all of the structural elements which influence the bay size. Foundation considerations may significantly effect results. An emphasis on cost has the consequence of introducing very subjective elements into the analysis process. The material and labor costs vary from supplier to supplier within a geographic region, and certainly among suppliers or subcontractors nationally. The approach is rational, yet it is intimately tied to those unit costs tabulated in *Table 1*.

PROCEDURES

The framing system considered is shown in Fig. 1. It consists of steel bar joists spaced nominally 6 ft o.c., supported by continuous hot-rolled steel girders spliced at the continuity inflection points. The girders, in turn, are supported by hot-rolled steel columns supported on a foundation system. The elements considered in the analysis are the size and spacing of the joists, as dictated by strength require-

TABLE 1

UNIT COST DATA

Foundations:	
Isolated soil excavation	$ 8.00/cy
Isolated rock excavation	$ 45.00/cy
Isolated compacted backfill	$ 12.00/cy
In-place footing concrete w/forms and reinforcing	$200.00/cy
In-place pile cap concrete w/forms and reinforcing	$200.00/cy
In-place caisson concrete	$175.00/cy
In-place pier concrete w/forms and reinforcing	$225.00/cy
Superstructure materials:	
Structural shapes (exclusive of shape and length extras)	$ 0.24/lb.
Structural accessories (plates and angles)	$ 0.30/lb.
Bolts (A325 ± 3″ lg.)	$ 0.65/ea
Steel joists (built-in cost data) min.	$ 0.29/lb.
max.	$ 0.37/lb.
Paint (shop coat)	$ 6.00/gal.
Superstructure labor:	
Fabrication labor (including overhead)	$ 30.00/hr
Erection labor (1 ironworker)	$ 37.50/hr
Crane cost	$ 81.00/hr
Miscellaneous:	
Sales tax	7½%
Profit	8%
Shop drawings	3½%

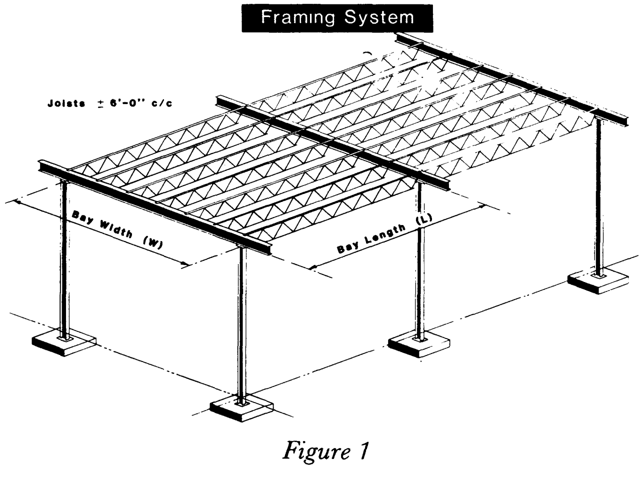


Figure 1

John L. Ruddy, P.E. is Vice President of Fletcher-Thompson, Inc., a Bridgeport, Connecticut structural engineering firm.

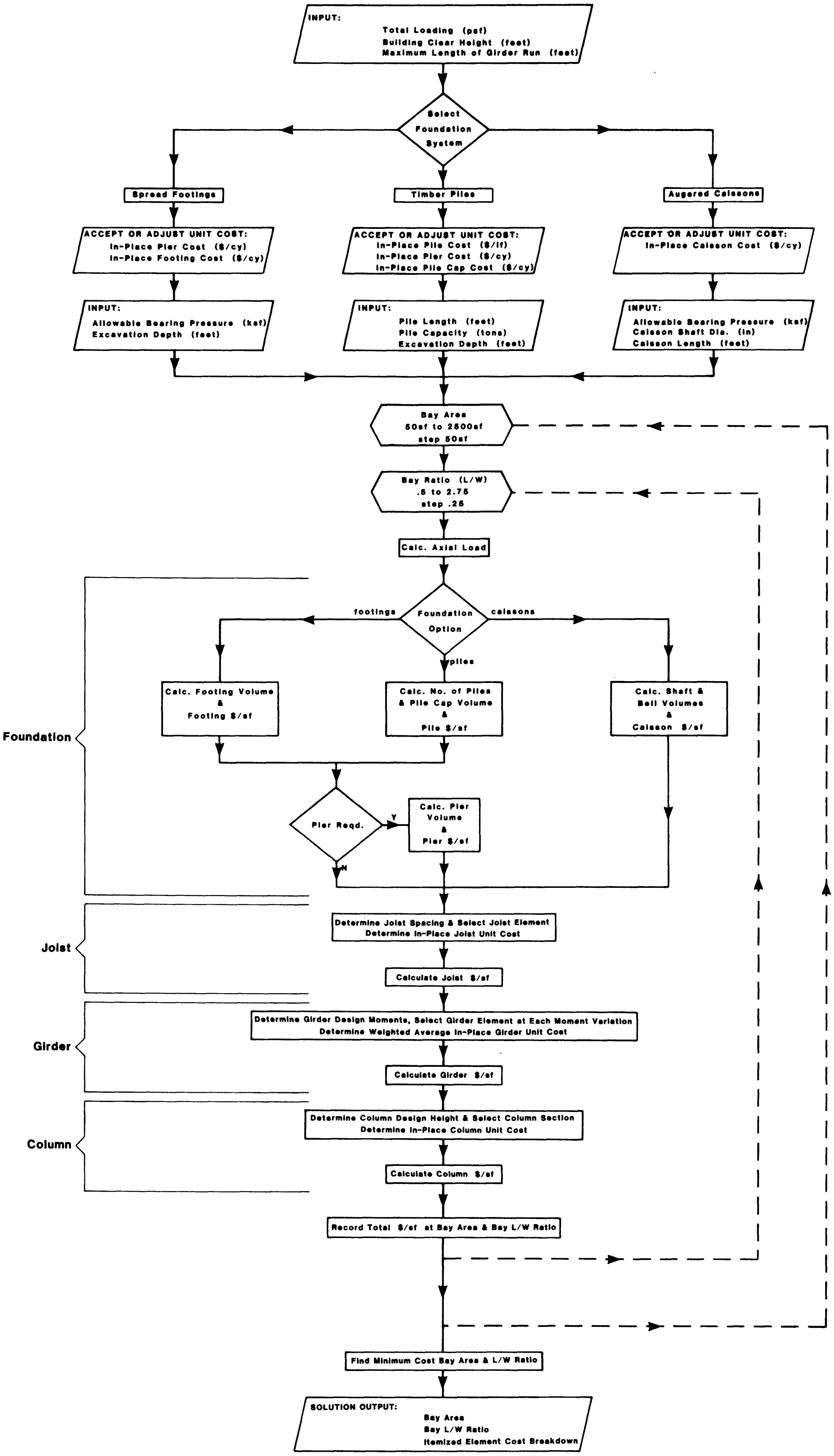

INPUT:
Total Loading (psf)
Building Clear Height (feet)
Maximum Length of Girder Run (feet)
Select Foundation System
Spread Footings
Timber Piles
Augered Caissons
ACCEPT OR ADJUST UNIT COST:
In-Place Pier Cost ($/cy)
In-Place Footing Cost ($/cy)
ACCEPT OR ADJUST UNIT COST:
In-Place Pile Cost ($/lf)
In-Place Pier Cost ($/cy)
In-Place Pile Cap Cost ($/cy)
ACCEPT OR ADJUST UNIT COST:
In-Place Caisson Cost ($/cy)
INPUT:
Allowable Bearing Pressure (ksf)
Excavation Depth (feet)
INPUT:
Pile Length (feet)
Pile Capacity (tons)
Excavation Depth (feet)
INPUT:
Allowable Bearing Pressure (ksf)
Caisson Shaft Dia. (in)
Caisson Length (feet)
Bay Area
50sf to 2500sf
step 50sf
Bay Ratio (L/W)
.5 to 2.75
step .25
Calc. Axial Load
Foundation Option
footings
caissons
piles
Calc. Footing Volume & Footing $/sf
Calc. No. of Piles & Pile Cap Volume & Pile $/sf
Calc. Shaft & Bell Volumes & Caisson $/sf
Pier Reqd.
Y
N
Calc. Pier Volume & Pier $/sf
Foundation
Determine Joist Spacing & Select Joist Element
Determine In-Place Joist Unit Cost
Calculate Joist $/sf
Joist
Determine Girder Design Moments, Select Girder Element at Each Moment Variation
Determine Weighted Average In-Place Girder Unit Cost
Calculate Girder $/sf
Girder
Determine Column Design Height & Select Column Section
Determine In-Place Column Unit Cost
Calculate Column $/sf
Column
Record Total $/sf at Bay Area & Bay L/W Ratio
Find Minimum Cost Bay Area & L/W Ratio
SOLUTION OUTPUT:
Bay Area
Bay L/W Ratio
Itemized Element Cost Breakdown

Figure 2

ments, the size of the girder as dictated by strength requirements, the size of the column as dictated by axial load requirements and the foundation requirements again dictated by axial load requirements.

A computer program has been developed to increment the bay area in 50-sq ft increments over a range of 2,500 sq ft to 50 sq ft, and to vary the length-to-width ratio (length being the span of the joists, width the span of the girders) at a specific area from a ratio of 0.5 to a ratio of 2.75 at an increment of 0.25. A design sequence considers 50 bay areas, each bay area being investigated at 10 length-to-width ratios. The process followed is indicated graphically by the flow chart in Fig. 2.

Three basic initial building parameters are input to the computer. These include: the building clear height, the maximum allowable length of girder run, and the total load to which the system is subjected. Three foundation options are available: spread footings, timber pilings and augered caissons. Additional data is requested after the appropriate foundation system is selected. Input requirements and cost determination procedures are described in the following:

Spread Footings—An allowable soil-bearing pressure and a depth of excavation is input under this option. The axial load is determined by multiplying the input total loading by the appropriate bay area. The axial load in conjunction with the input allowable soil-bearing pressure determines an area requirement for a spread footing. This area requirement is satisfied using a square footing considering plan dimension increments of 6 in. The footing thickness is determined considering punching shear, assuming concrete having a compressive strength (f_c') of 3,000 psi. The footing thickness increment is stepped at 2 in. until punching shear requirements are satisfied. The minimum footing plan dimension considered was 2 ft-6 in., and the minimum footing thickness considered was 12 in.

Excavation volumes were determined assuming that at the bearing elevation, the excavation plan dimensions were 2 ft greater than the footing plan dimension (1 ft beyond the footing in all directions). The side slopes of the excavation were assumed to occur at one unit horizontal to one unit vertical. Those assumptions in conjunction with the input depth of excavation allow a volume calculation regarding excavation. Deducting the footing volume from this excavation volume revealed the backfill volume requirements.

Using the unit cost for excavation backfill and foundation concrete as indicated in *Table 1*, a cost per footing is determined. This cost is then divided by the bay area to reveal the foundation cost per sq ft. Figure 3 shows a plot of the foundation cost variation over the bay areas considered for the specific condition of a 55 psf total load, a 4 ksf allowable bearing pressure and a depth of excavation of 2 ft-6 in. This graph reveals the consequences of the finite sizing of the foundation element. Situations where the structural solution very closely matches the structural requirements are more economical than those which are on the verge of an increment reduction.

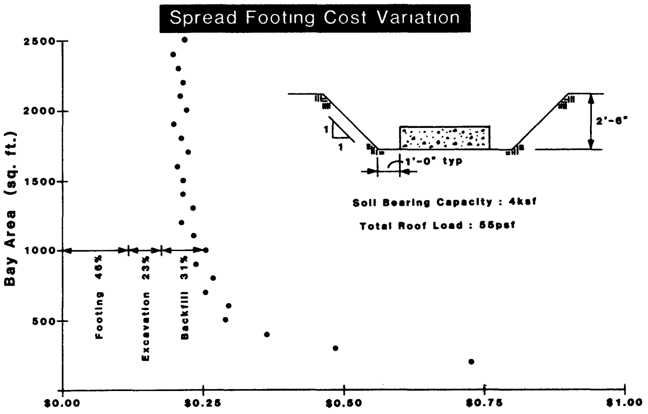


Figure 3

Timber Piles—Additional input requirements under this foundation option include the individual pile capacity and an estimated length of pile, as well as the excavation depth required for the pile cap. The actual load is again determined by multiplying the bay area by the input total loading. The computer has stored in its memory the capacity of 15-ton, 25-ton, 35-ton and 45-ton piles for pile groups consisting of two through nine piles.

The stored capacities reflect efficiency reductions dictated by cluster effects. The stored pile capacity which is closest to, but exceeding, the input pile capacity is used in the analysis. And it is this vector of capacities which is used to determine the pile count. Associated with each pile capacity and grouping is a pile cap volume. This information is also stored in the computer and the appropriate value retrieved.

Excavation and backfill quantities are determined in the same manner as described under "Spread Footings." The total foundation cost is calculated using the determined volumes of excavation, backfill and pile cap as well as the linear foot of pile in conjunction with the unit material costs as tabulated in *Table 1*. This total cost is again divided by the bay area to reveal the foundation cost per sq ft.

Figure 4 details the variation of the cost per sq ft of a pile foundation solution for the specific case of 30-ft long, 20-ton capacity piles subjected to a total load of 60 psf. The graph very significantly indicates the cost advantages of dropping from a four-pile group requirement to a three-pile group requirement and subsequently to a two-pile group requirement.

Augered Caissons—Additional input requests under this option include a caisson shaft diameter, an allowable bearing pressure and an estimated length of caisson. The axial load is again determined by multiplying the input total load by the bay area, and to this is added a caisson weight. Caisson weight is determined assuming a net added unit density of 50 pcf. This unit density is a conservative

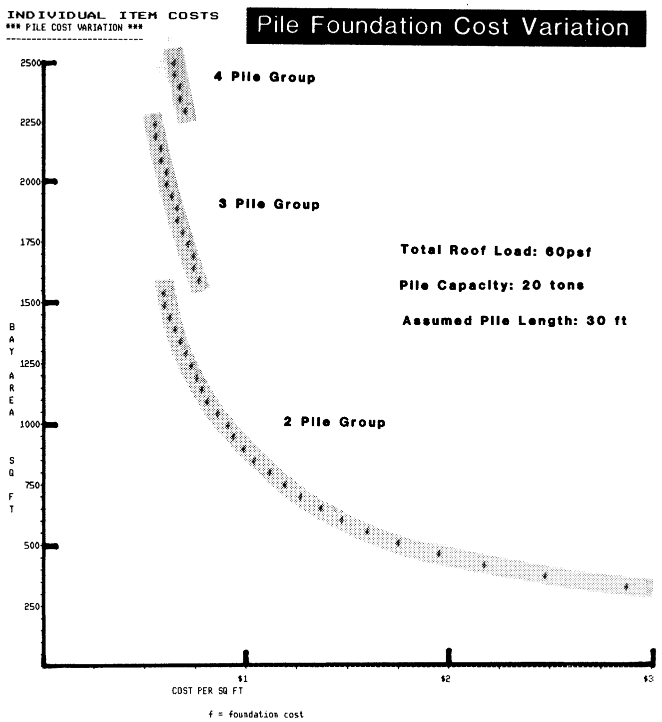


Figure 4

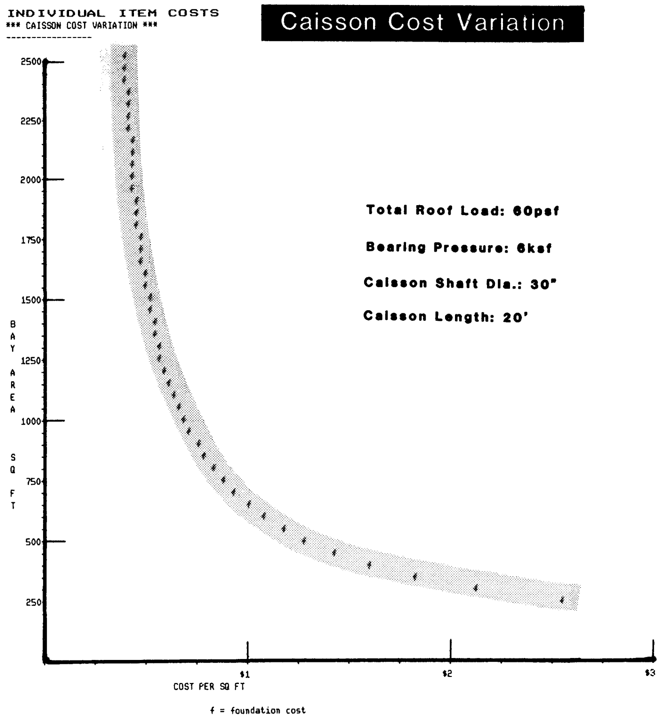


Figure 5

approximation to the difference in weight of soil removed vs concrete placed.

The bearing area provided by the shaft is compared to the required bearing area as dictated by the input allowable soil-bearing pressure. If bearing area requirements are met by the shaft independently, the shaft volume is determined. If the shaft area is inadequate, employment of a bell is assumed. The bell diameter is determined by adding successive 2-in. increments to the shaft diameter until the bearing pressure is below the allowable bearing pressure input. The bell volume and shaft volume are then used in conjunction with the unit cost for caissons to determine the total cost of this element. No excavation or backfill requirements are considered under this option. The total cost of this foundation solution is again divided by the appropriate bay area to reveal the cost per sq ft under this option.

Figure 5 indicates the cost variation over the incremented bay area for the specific condition of a 20-ft long, 30-in. diameter shaft caisson subjected to a total load of 60 psf. This curve is much smoother than the previous two, reflecting the smaller increment of design solutions which are able to respond to design requirements.

Steel Joists—The bay area, in conjunction with the bay ratio, is used to determine the width and length under investigation. Steel joists are assumed to be evenly spaced over the bay width, starting at a column center line and ending at column center line. This spacing is such that the center-to-center dimension of the joist does not exceed 6 ft-6 in. The basis for this restriction is that $1\frac{1}{2}$-in. × 22-ga. roof deck is assumed, and requirements directed by *Factory Mutual Engineering Guidelines* are to be met.

Those restrictions provide sufficient stiffness of the deck element so that foot traffic over the roof will not result in a bond break of the roofing membrane. The resultant spacing is used in conjunction with the input total load and the calculated span of the roof joists to determine a bending moment. The tabulated capacities presented by the Steel Joist Institute for H Series and LH Series Joists were converted to equivalent moment capacities. Using the moment capacity vs weight as a basis, the range of joist sizes available was reduced to 36 sections and those are indicated in *Table 2.*

The sections available reflect the maximum capacity for least weight. Using this matrix of capacity vs weight in conjunction with the required moment capacity, a joist foot-weight is determined. This foot-weight, when divided by the determined spacing, reveals the weight of the joists per sq ft of area.

Figure 6 is a graphic representation of this weight variation over the bay area. It again reflects the consequence of having a joist capacity which very nearly matches the required capacity vs having reserved capacity available as a consequence of the finite selections available. Joist costs were assumed to be composed of three elements: material costs, erection labor costs and crane involvement costs. Material costs generally decrease for H Series Joists from a maximum at the lighter foot-weight sections to a minimum at the heavier foot-weight sections. The reverse is true for LH Series Joists, which reflect a material cost increase as the foot-weight increases.

TABLE 2

JOIST MATRIX

Section Designation	Mom. Cap. (kips/ft)	Weight (lbs.)	Material (cents/lb.)	Ironworker (min.)	Crane (min.)
10 H 3	9.67	5.00	0.370	46.67	4.33
12 H 3	11.67	5.20	0.360	46.67	4.33
14 H 3	13.75	5.50	0.360	48.33	4.33
12 H 4	15.00	6.20	0.350	48.33	4.33
14 H 4	17.67	6.50	0.350	52.50	4.33
16 H 4	18.42	6.60	0.350	52.50	4.33
14 H 5	21.58	7.40	0.340	59.17	4.33
16 H 5	24.08	7.80	0.330	59.17	4.33
18 H 5	27.08	8.00	0.330	69.17	5.67
20 H 5	30.42	8.40	0.320	70.83	5.67
18 H 6	31.92	9.20	0.320	74.17	5.67
20 H 6	33.83	9.60	0.320	80.00	6.67
22 H 6	35.17	9.70	0.320	84.17	7.50
24 H 6	38.50	10.30	0.310	84.17	7.50
22 H 7	43.83	10.70	0.310	87.50	7.50
24 H 7	48.00	11.50	0.310	95.83	9.17
22 H 8	54.42	12.00	0.310	95.83	9.17
24 H 8	59.67	12.70	0.310	95.83	9.17
26 H 8	65.33	12.80	0.310	95.83	9.17
28 H 8	70.50	13.50	0.310	95.83	9.17
30 H 8	75.75	14.20	0.300	95.83	9.17
28 H 9	83.33	15.20	0.300	100.00	10.00
30 H 9	89.58	15.40	0.300	100.00	10.00
28 H 10	93.67	16.80	0.300	115.10	11.67
30 H 10	100.58	17.30	0.300	115.10	11.67
28 H 11	108.33	18.30	0.295	125.00	11.67
30 H 11	116.42	18.80	0.295	125.00	11.67
40 LH 08	132.80	20.00	0.295	140.00	13.33
48 LH 10	200.00	25.00	0.310	144.17	14.17
48 LH 11	217.00	27.00	0.310	144.17	14.17
48 LH 12	274.00	31.00	0.320	171.67	18.33
48 LH 13	328.00	35.00	0.330	190.83	20.83
48 LH 14	387.00	36.00	0.340	201.67	21.67
48 LH 15	445.00	41.00	0.340	203.33	23.33
48 LH 16	513.00	47.00	0.350	211.67	25.00
48 LH 17	576.00	54.00	0.350	218.33	25.00

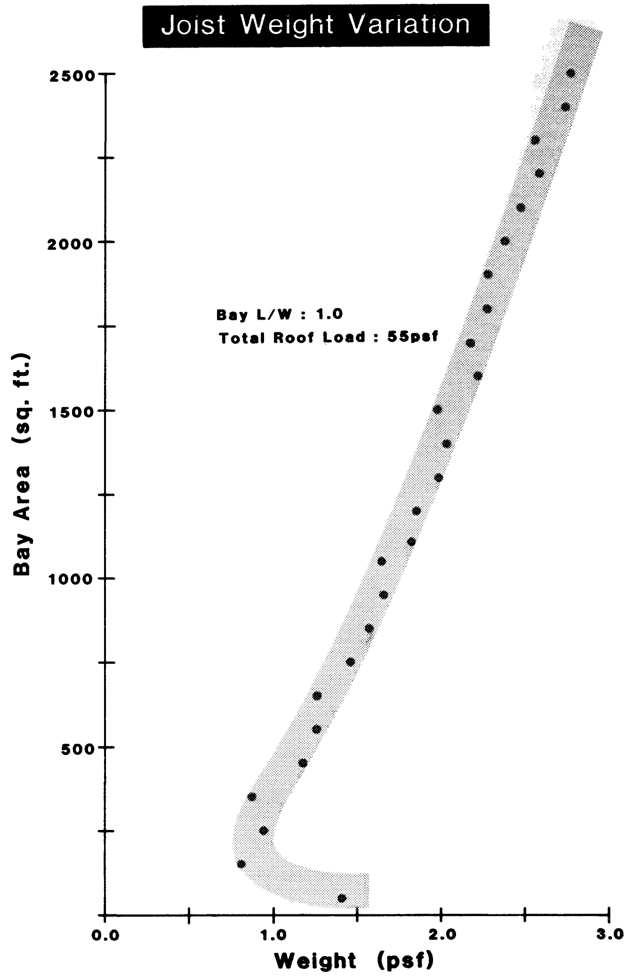


Figure 6

The material cost assumptions are tabulated in *Table 2* for only those joist sections considered. The labor involvement requires unloading, erecting, spreading and bridging the joists. Lighter sections, in the range of 300 lbs per joist, are able to be unloaded and erected in bundles and hand-spred, an operation which consumes less field labor time than individual unloading erection and placement. This condition is reflected in the time estimates tabulated. A crew consisting of a foreman and four ironworkers was assumed, and time records associated with a specific joist size were tabulated. Those time records, divided by the five personnel involved, result in a labor-time requirement per ironworker per joist. Those time assumptions are tabulated in the joist matrix in *Table 2.*

Similarly, crane time requirements per joist were assembled and those times are those also tabulated in *Table 2.* Consequently, a specific joist cost is determined by multiplying the weight of the element by the unit cost from the matrix plus the ironworker time multiplied by the unit ironworker labor cost, plus the crane time multiplied by the unit crane cost. If this cost is divided by the total joist weight an in-place cost per pound for the joist element is determined.

A graph of this variation over the bay area is presented in Fig. 7. Each point on the ordinate of the graph reflects a specific span as well as a specific joist spacing for the condition of square bays subjected to a total roof load of 55 psf. The in-place cost of the joist elements increases as the bay size decreases. Cost variation, as a function of bay area, is the product of the weight curve and the unit cost curve. The cost variation is shown graphically in Fig. 8.

Steel Girders—Steel girder selections are made assuming that uniformly distributed loading is applied to a system of cantilevered beams. The magnitude of the load is calculated by multiplying the joist span by the input total load yielding a load magnitude per foot of girder length. Design moments are a function of the number of spans contributing to the continuity. That span count is determined by dividing

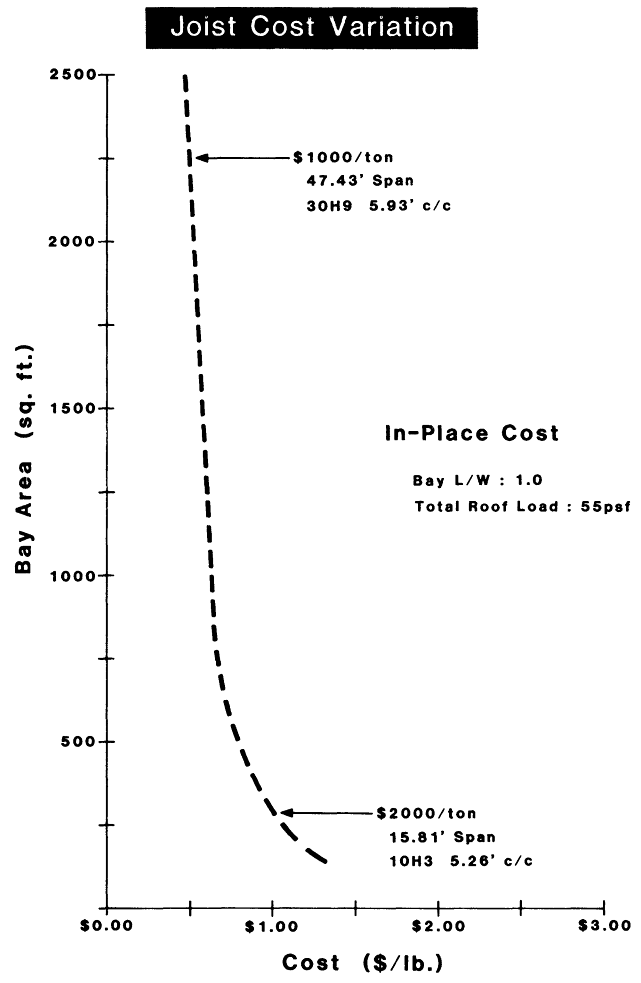


Figure 7

the input maximum length of girder run by the bay width. The integer portion of that calculation is assumed to adequately account for continuity effects in the girder design. The length and governing moment of individual members in the girder run vary over the length of the run.

The moment coefficient and length, as well as the percent of the total run governed by the coefficient and length, are stored in the computer for two, three, four and five spans. For situations allowing greater than five spans, the coefficients, lengths and percents of run are also stored for even or odd span counts. The data used is tabulated in Fig. 9. The design moment for each member in a girder run is determined by multiplying the moment coefficient by the load magnitude per foot of girder run by the bay width squared. Member selections are made for each variation in moment coefficient within the girder run using the economy sections only. Moment capacities of the economy sections at an unsupported length of 6 ft are compared with design moment requirements, and that section which is the first to provide a moment capacity which exceeds the design requirement is selected.

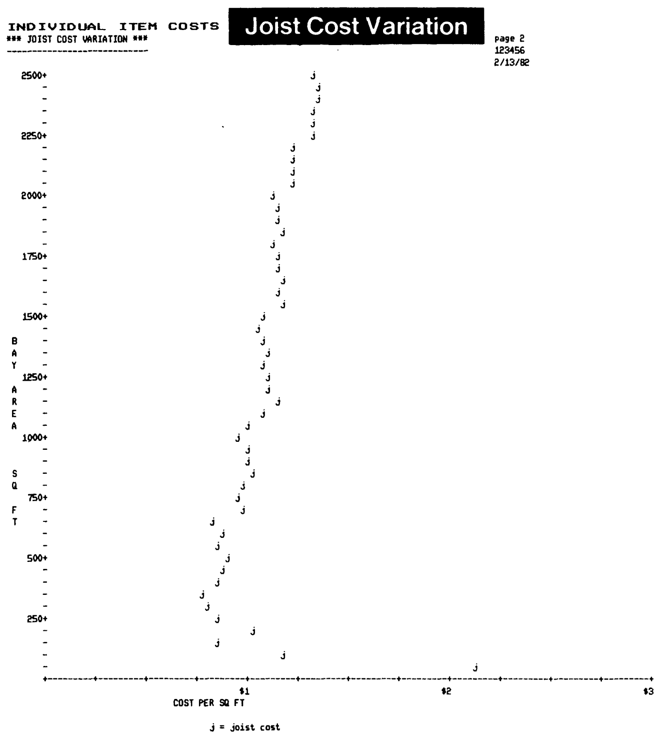


Figure 8

Structural steel is assumed to conform to A36 requirements, and moment capacities reflect a 24-ksi bending stress. If the percent of the total run which is controlled by an individual member is used in conjunction with that member foot-weight, a weighted average psf can be calculated. That variation of girder weight vs bay area for the specific case of square bays, a total roof load of 55 psf, and the available length of girder run of 150 ft is shown in Fig. 10. The computer program does not perform this exercise, but determines a weighted average cost psf for the girder elements.

Girder costs are composed of three elements: material, fabrication and erection costs. There is a significant in-place unit cost differential between shallow members with lighter

Girder Design Parameters

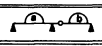
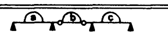
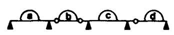
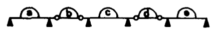
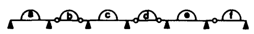
No. of Spans	Schematic	Moment Coeff.	Member	Length	% of Run
2		.086	a	1.172 x L	59
		.086	b	0.828 x L	41
3		.086	a,c	1.220 x L	81
		.039	b	0.560 x L	19
4		.086	a	1.204 x L	30
		.051	b	0.639 x L	16
		.063	c	1.282 x L	32
		.096	d	0.875 x L	22
5		.086	a,e	1.204 x L	48
		.051	b,d	0.639 x L	26
		.063	c	1.314 x L	26
≥6 even		.086	a	1.204 x L	20
		.051	b	0.639 x L	11
		.063	c	1.304 x L	22
		.063	d	0.706 x L	12
		.063	e	1.272 x L	21
		.096	f	0.875 x L	14
≥7 odd		.086	a,g	1.204 x L	35
		.051	b,f	0.639 x L	18
		.063	c,e	1.304 x L	37
		.063	d	0.706 x L	10

Figure 9

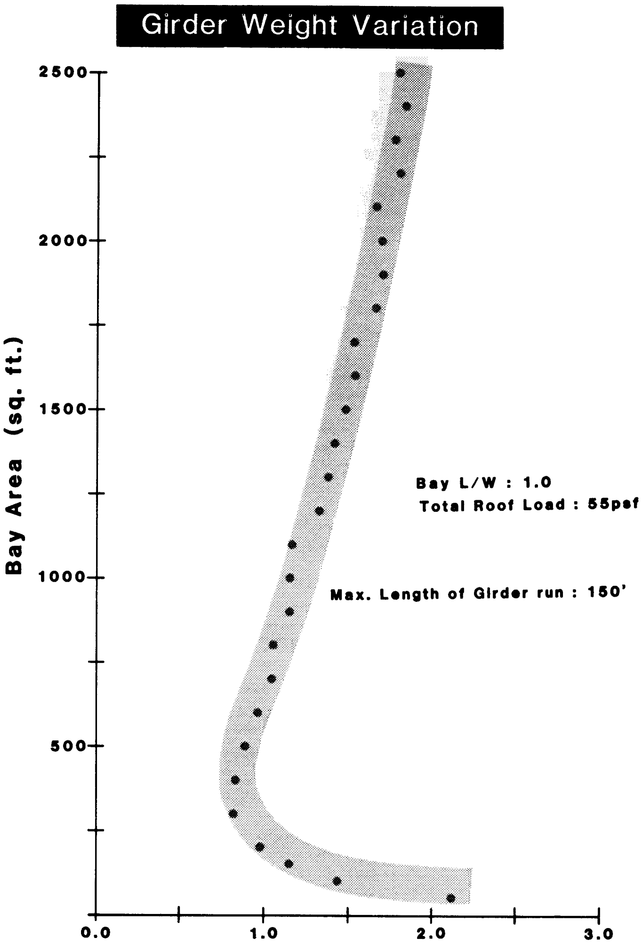


Figure 10

Girder Cost W12x26 - 25' lg

Item	Quant	Unit Cost	Mat. Cost	Quant	Unit Cost	Fab. Cost	Quant	Unit Cost	Erect Cost
W12x26	650#	.270	175.50						
4-L's 4x4x5/16	15#	.30	4.50						
Paint	.33gal	6.00	2.00						
Bolts	4-3/4	.65	2.60						
Tax (7 1/2%)			13.85						
Shop Dwg						12.00			
Fabrication				1.5hr	30.00	45.00			
Paint & Load				0.6hr	30.00	18.00			
Erect & Bolt							.15hr	206.00	30.90
Sub-total			198.45			75.00			30.90
8% Profit			15.88			6.00			2.47
			214.33			81.00			33.37

Material: $214.33 (65%)
Fabrication: $81.00 (25%)
Erection: $33.37 (10%)
$328.70

$988.57/ton

Figure 11

Girder Cost W33x118 - 45' lg.

Item	Quant	Unit Cost	Mat. Cost	Quant	Unit Cost	Fab. Cost	Quant	Unit Cost	Erect Cost
W33x118	5310#	.2625	1393.88						
4-L's 4x4x3/8	48#	.30	14.40						
Paint	2.7gal	6.00	16.20						
Bolts (3/4")	10	.65	6.50						
Tax (7 1/2%)			107.32						
Shop Dwg						81.25			
Fabrication				5.8hr	30.00	174.00			
Paint & Load				4.6hr	30.00	138.00			
Erect & Bolt							.31hr	206.00	63.86
Sub-total			1538.30			393.25			63.86
8% Profit			123.06			31.46			5.11
			1661.36			424.71			68.97

Material: $1661.36 (77%)
Fabrication: $424.71 (20%)
Erection: $68.97 (3%)
$2155.04

$804.42/ton

Figure 12

foot-weights and the deeper members with the heavier foot-weights. Figures 11 and 12 indicate this differential. These figures not only indicate the unit cost differential but also reflect the operations undertaken by the computer in arriving at a member cost. Figure 11 is used as an example of the procedures followed to arrive at a member cost. The figure reflects a cost estimate for a W12x26 girder, 25-ft long. Material consists of the W12x26, four angles 4 X 4 X $5/16$, paint, bolts and a tax on all the purchased items.

The unit material cost is a function of a basic product cost which is adjusted by shape and length extras. The base material price is a requested input value. The shape extras associated with those economy sections considered are stored in a girder matrix, as tabulated in *Table 3.* Length extras are determined in accordance with The Bethlehem Steel Corporation, Sept. 27, 1981, values. The base price, plus extras, is multiplied by the total weight of the element to arrive at a material cost. Standard connection clip angles are assumed for each of the economy sections considered and the weight associated with those items is also stored in the girder matrix, again those tabulated in *Table 3.* Those clip weights are retrieved and multiplied by an input unit cost for accessories.

Paint quantities are assumed to be one gallon per ton of structural steel required. The base unit price for paint is multiplied by that calculated paint quantity. Also associated with each of the sections considered is a bolt count, the final item tabulated in the girder matrix. The appropriate bolt count is multiplied by the unit bolt cost. The sum of all the preceding cost items is multiplied by a sales tax rate, and the sum of the material costs plus tax is escalated by 8% to afford a profit, in this specific instance revealing $214.33 for material.

Fabrication labor costs, including painting and loading, were tabulated and least square curve-fitting techniques were used to determine a fabrication time equation as a function of member foot-weight and length. The 2.1 hours of fabrication time indicated for the W12x26 and the 10.4 total fabrication hours indicated for the W33x118 are representative of the times predicted by the equations built into the computer. Those fabrication times are multiplied by a fabrication labor rate to arrive at a shop fabrication

TABLE 3

GIRDER MATRIX

Depth (in.)		Ft-Wt (lbs.)	Mom. Cap. (kips/ft)	Shape Extra (cents)	Clip Wt (lbs.)	Bolts (number)
8	×	13	18	3.50	14.12	8
12	×	14	24	3.50	21.82	12
12	×	16	27	3.50	21.82	12
12	×	19	35	3.50	21.82	12
14	×	22	51	2.25	21.82	12
16	×	26	70	2.00	21.82	12
14	×	30	84	2.00	21.82	12
16	×	31	86	2.00	21.82	12
18	×	35	115	2.00	29.52	16
18	×	40	137	2.00	29.52	16
21	×	44	163	2.25	29.52	16
21	×	50	189	2.25	29.52	16
24	×	55	228	2.25	29.52	16
24	×	62	262	2.25	29.52	16
24	×	68	308	1.75	29.52	16
24	×	76	352	1.75	29.52	16
27	×	84	426	1.75	37.22	20
27	×	94	486	1.75	37.22	20
30	×	99	538	2.00	37.22	20
30	×	108	598	2.00	37.22	20
30	×	116	658	2.00	37.22	20
33	×	118	718	2.25	44.92	24
33	×	130	812	2.25	44.92	24
36	×	135	878	2.60	44.92	24
33	×	141	896	2.25	44.92	24
36	×	150	1010	2.60	44.92	24
36	×	160	1080	2.60	44.92	24
36	×	170	1160	2.60	44.92	24
36	×	182	1250	2.60	44.92	24
36	×	194	1330	2.60	44.92	24
36	×	210	1440	2.60	44.92	24
36	×	230	1670	2.60	44.92	24
36	×	245	1790	2.60	44.92	24
36	×	260	1910	2.60	44.92	24
36	×	280	2060	2.60	44.92	24
36	×	300	2220	2.60	44.92	24

labor cost. Also in this category, fabrication, is included a cost estimate for shop drawings. The computer assumes 3½% of the total cost for this item. The sum of those fabrication costs are again escalated by 8% to afford a profit, in this specific example, resulting in an $81 cost for fabrication.

A similar curve-fitting technique was used to estimate the erection labor time. The unit erection costs were assumed to require a foreman and four ironworkers, plus 25% of the hourly crane cost. The product of the time estimate and the unit erection crew labor cost reveals an erection cost which is again escalated by 8%. The specific example reveals a $33.37 erection expenditure. The total of these costs in this case is $328.70, which represents an in-place cost for a W12x26 by 25-ft long of $988.57 per ton.

An immediate observation is that the largest percentage of the in-place cost is that of the material, and the least significant the erection costs. Also of significance, is the relative in-place cost differential between a W12x26 and a W33x118. This cost procedure is used for each of the members selected in the girder run. The cost for each element is multiplied by the appropriate percentage of run to arrive at a weighted unit cost for the girders. Figure 13 indicates the cost variation as determined by this method. Each point on the ordinate of the graph reflects a specific span count, span length and multiple member selections.

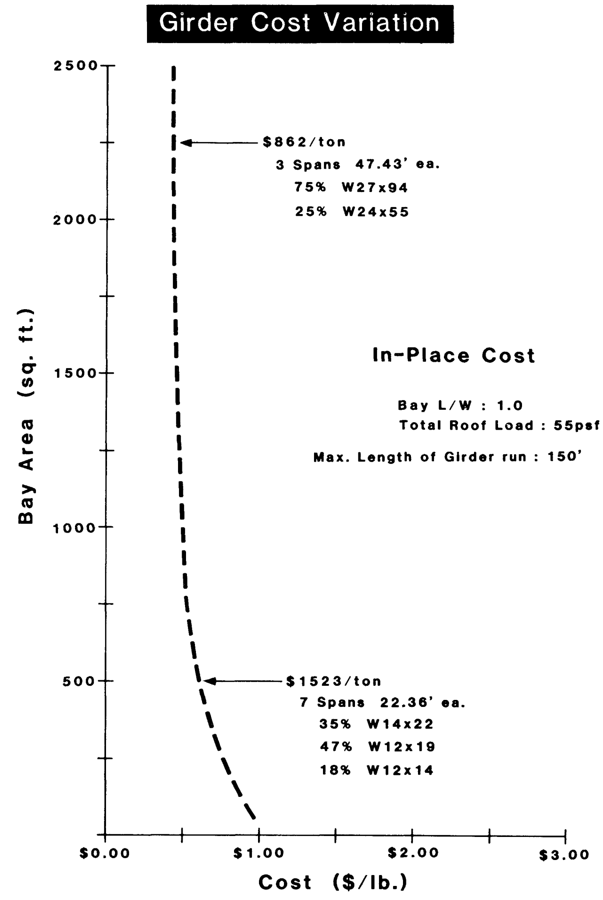


Figure 13

Steel Columns—The columns are the last elements to be considered in the bay cost. The columns are assumed to be subjected to axial load only. That load is determined by multiplying the input total loading by the appropriate bay area. The column design height is calculated on the basis of the input clear height requirements in conjunction with the joist and girder depths which were selected at the specific area and bay length-to-width ratio. The column design height is the dimension from the finished floor to the top of the steel girder. This dimension is determined by adding to the clear height the depth of the girder for situations where the bottom of the joists does not fall below the bottom of the girders.

If the joists control the clear height, the column design height is the input clear height plus the girder depth plus the distance between the bottom of the joist and the bottom of the girder. Only hot-rolled column sections are considered, and the range of selections is W8x24 through W8x39, W10x45 through W10x100 and W12x40 through W12x210. A vector of load capacities for each column section is stored in the computer for 2-ft height increments from a minimum of 12-ft to a maximum of 38 ft.

The calculated axial load and clear height are used to access those vectors and the least foot-weight section which will provide axial load resistance at the calculated design is selected. The actual length of the column used to calculate the total weight of this element is the distance from the underside of the girder to the finished floor, plus the column extension below the floor. If the calculated foundation thickness is subtracted from the input depth of excavation, the dimension between the finished floor and top of the foundation results. If this dimension is 2 ft or less, the column length is calculated as the dimension from the top of the foundation to the underside of the girder. If the dimension between the finished floor and the top of the footing exceeds 2 ft, a concrete pier is assumed, so the column extension below the floor is 1 ft (a pier less than 1 ft in height is not used). If the preceding calculations reveal that a pier is to be employed, the cost of the pier is added to the foundation cost.

The procedures used to determine in-place column costs are very similar to those employed for girders. The total material employed is assumed to be the column, a cap plate and a base plate. Standard plates were determined for each column section and the weight of these plate elements is stored in the computer. The input unit steel price is again escalated by the appropriate shape and length extra and this unit cost is multiplied by the total column weight. To this material cost is added the product of plate weight, and the unit cost for accessory materials.

Paint quantity is again assumed to be one gallon per ton of material, and the cost for this element is that quantity multiplied by the paint material unit cost. The sum of the material cost is multiplied by a tax rate and that material cost is escalated by 8% to afford a profit. Fabrication time and erection time observations were used to determine least square fit equations for these times and fabrication and erection costs were determined using these time approximations multiplied by the unit labor cost. Fabrication and erection costs are also increased by 8% for profit.

The subsequent total in-place cost was again increased by $3\frac{1}{2}$% to approximate shop drawing expenditures. The weight variation as a function of bay area for the specific case of a 55 psf total load and a building clear height of 15 ft is shown in Fig. 14. The column unit cost, which would be associated with the Fig. 14 weight variation, is shown in Fig. 15. The product of these two curves would reflect the in-place cost psf of columns as a function of bay area.

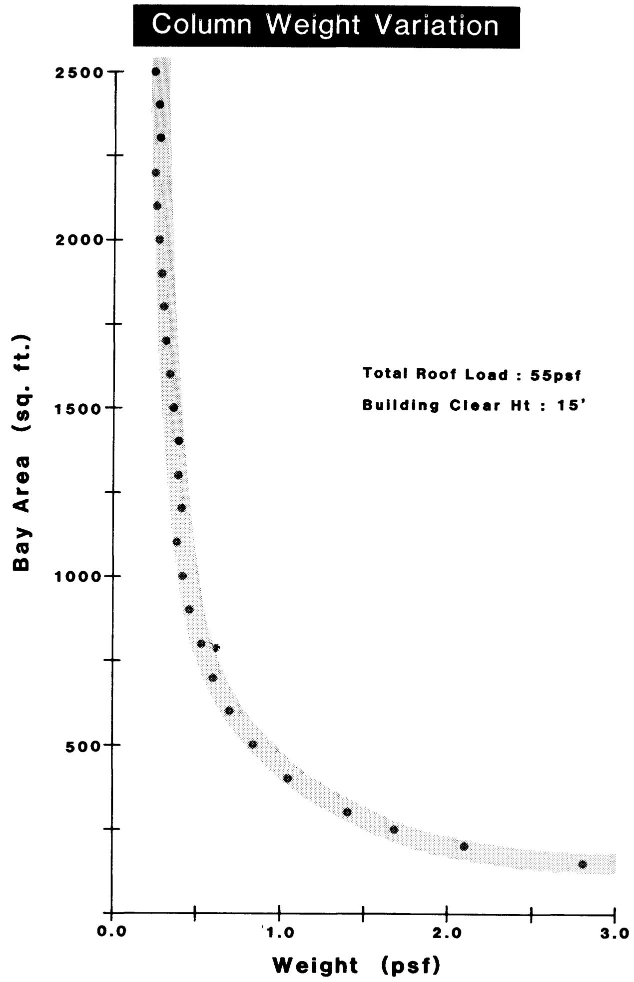


Figure 14

Least Cost Bay Area and Length to Width Ratio—In-place cost is assumed to be the sum of the foundation cost, the joist cost, the girder cost and the column cost. That sum is recorded at each area and length-to-width ratio variation resulting in a vector of 500 costs. The minimum value in that vector is found and the associated area and bay ratio is output as the least cost configuration.

RESULTS

Representative results for a building subjected to a 55 psf roof loading, a 20-ft building clear height requirement, a spread footing foundation solution with a 4 ksf bearing capacity, an excavation depth of 2 ft-6 in., and a maximum permissible length of girder run of 200 ft is shown in Fig. 16. The costs at multiple bay ratio configurations are plotted at area increments of 100-sq ft. Acceptable solutions were assumed to occur over a bay area range of 500–1,500 sq ft. A square bay was revealed as a solution at only one increment over that range (600 sq ft). Bay ratios of 1.25, 1.5 and 1.75 in almost every instance proved more economical than the solution associated with a square bay. The

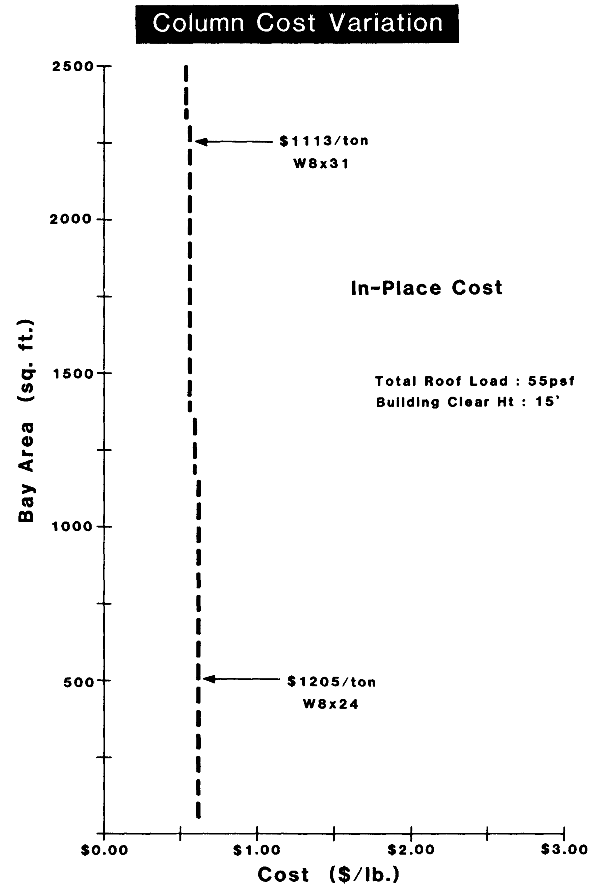


Figure 15

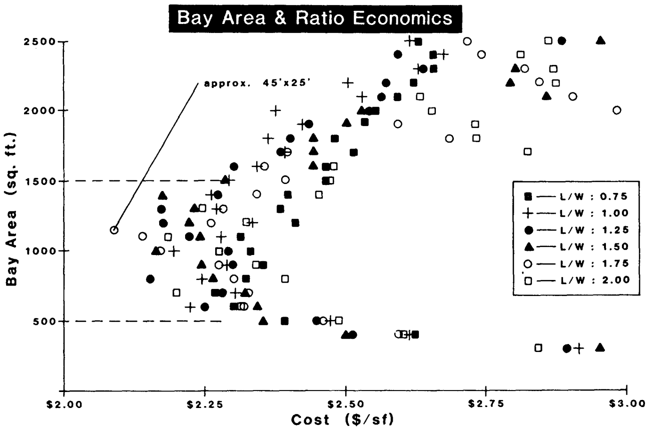


Figure 16

optimum solution at a bay area of 1,150 sq ft and a ratio of 1.75 reflects the condition where the design solution almost ideally matches the design requirements.

Figure 17 illustrates the consequences of an alternative approach to determining the least cost bay area and configuration. It reflects a study using the same building parameters as employed with Fig. 16 with the exception that the framing system is subjected to 60 psf in lieu of 55 psf, and the objective of the study was to find a bay area and configuration which would result in the least weight of the structural steel elements only. A least weight of 3.34 psf was calculated at a bay area of 450 sq ft and a length-to-width ratio of 1.25. Both increases and decreases from this predicted minimum weight solution result in significant weight increases.

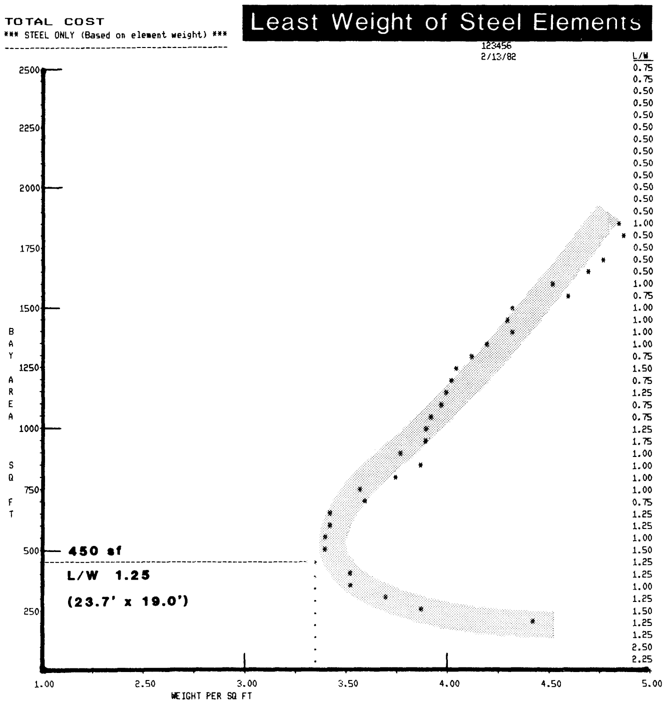


Figure 17

Figure 18 illustrates a solution to the same problem. However, in this case the objective is to determine the solution which represents the least cost of the steel elements. A significant increase in bay area results, and much less significant consequence due to divergence from this least cost solution is indicated.

Figure 19 represents a solution to the same problem. But in this instance, foundation considerations are added. The resulting solution is identical to that determined by investigating the least cost of the steel elements only. The entire curve is shifted to the right, reflecting the increase in cost inherent with adding the foundation expense. The variation in cost of shallow spread footings as a function of changes in bay area was not significant in comparison to the cost variations of the other elements considered; columns, girders and joists. Consequently, the inclusion of shallow spread-footing foundation costs in the overall analysis generally had the effect of revealing the same area and bay configuration as that which resulted when considering the cost of the steel elements only.

However, if subsurface conditions were such that a deep foundation solution was required, foundation considerations have a significant influence on the least cost bay. Figure 20 shows the consequences of employing augered

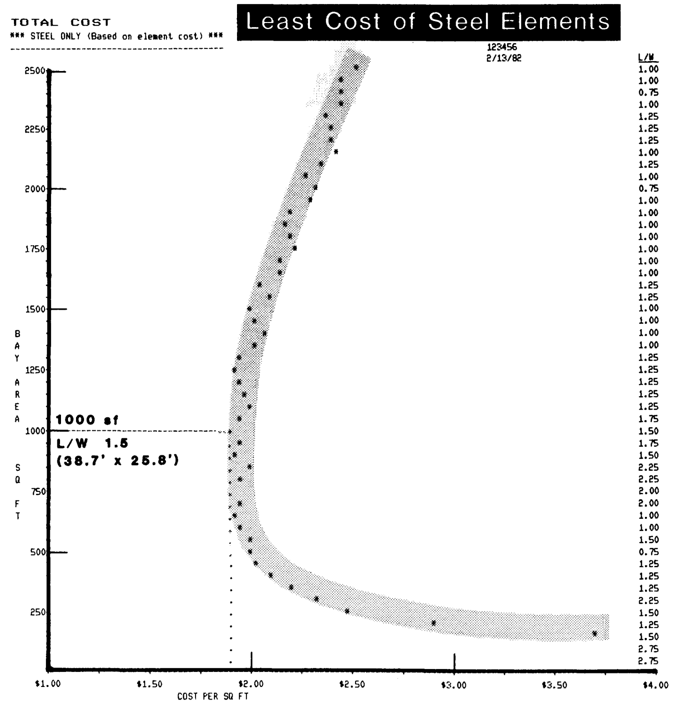


Figure 18

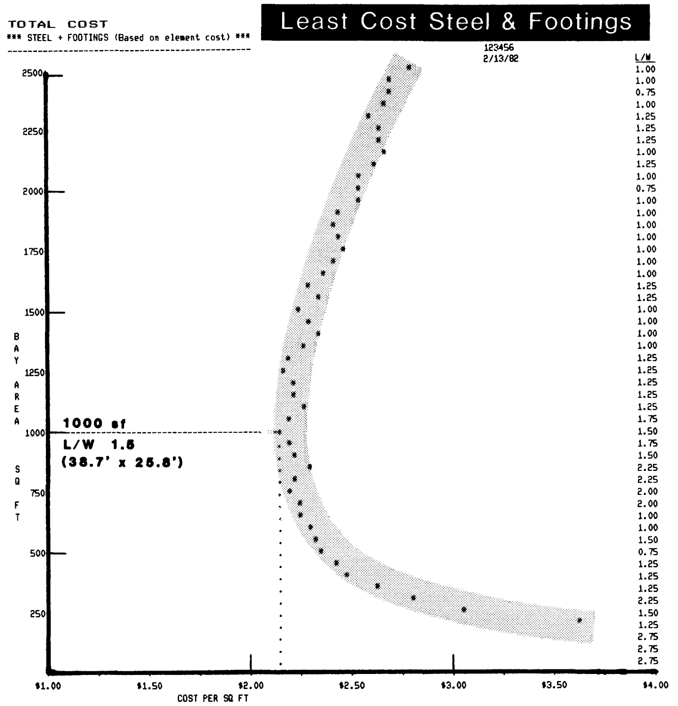


Figure 19

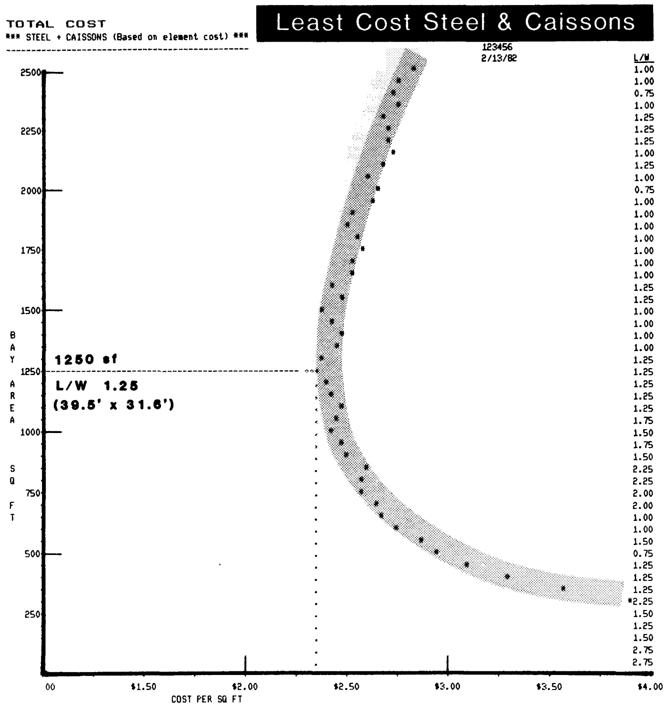


Figure 20

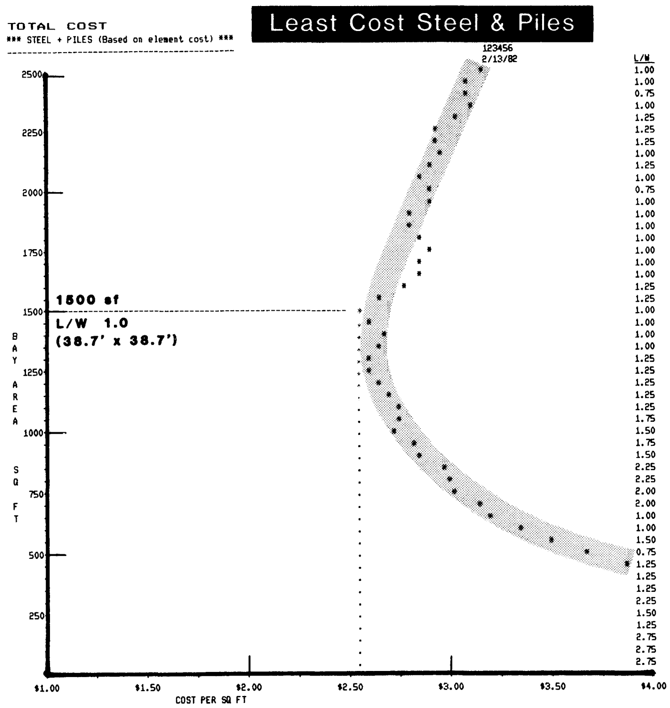


Figure 21

caissons to a depth of 15 ft, using a 30-in. shaft and a 6 ksf bearing capacity. The least cost bay in this situation is 1,250 sq ft at a length-to-width ratio of 1.25. Figure 21 details a solution where subsurface conditions required utilization of 20-ton, 30-ft long timber piles. In this instance, a least cost bay area of 1,500 sq ft at a square ratio was revealed.

Figure 22 illustrates the sq ft cost variation of various elements included in the analysis. This specific example represents shallow spread footing foundations. The cost

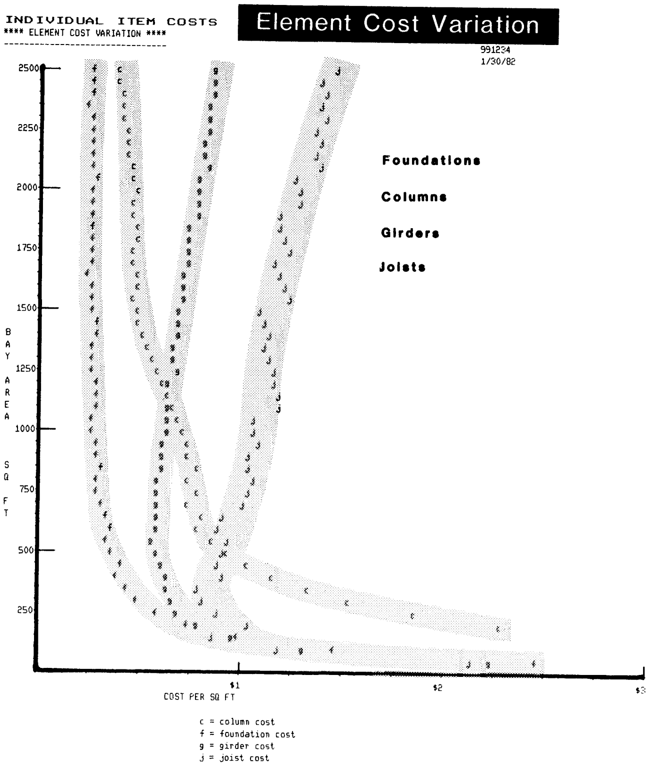


Figure 22

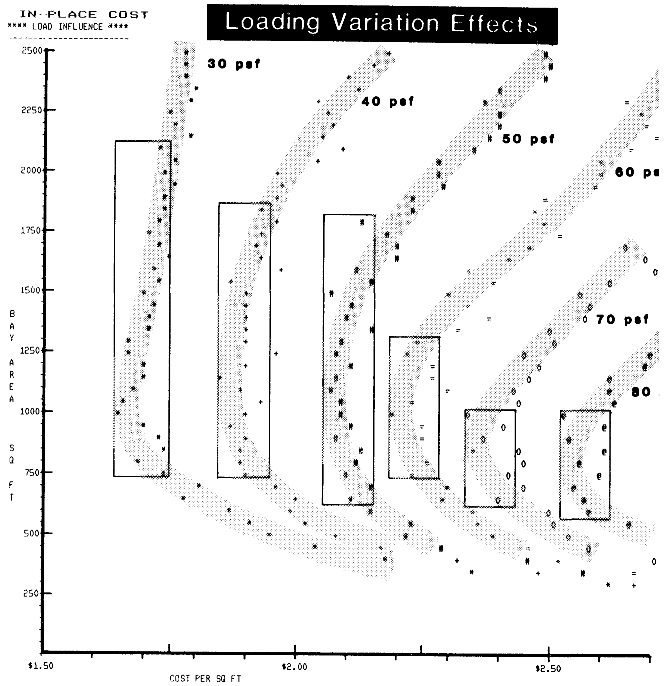


Figure 23

variation of the foundation in this case is relatively constant over the range of economical bay sizes. Obviously, in the bay area range of 750 to 1,500 sq ft, the effect of including this element is to shift the curve laterally, with only minor effects on the shape of the curve. This is not the case for deep foundations which reveal a much more rapid price increase with reductions in bay area than is revealed by the shallow footing solution. Therefore, deep foundation solutions have the effect of increasing the least cost bay area size.

In addition to the unit costs, the varibles considered in the analysis include total loading, building clear height, maximum length available for a run of girders, allowable bearing pressure in the case of spread footings in augered caissons and an allowable pile capacity in the case of timber piles. The effect of the alternative foundation systems was related in the preceding sections. In the case of spread footings and augered caissons, changes in allowable bearing pressure had little or no effect on the least cost bay area and bay ratio. The input maximum length of a girder run affords the computer the ability to include the effects of continuity in determining the bay size and configuration. Variations in this input parameter also had limited effects.

A simple span solution resulted in a bay area of 750 sq ft at a length to width ratio of 2. However, having a 300-ft length available to develop continuity resulted in a bay area of 1,000 sq ft at a length to width ratio of 1.5. Variations in the building clear height also did not affect the bay area or ratio which was revealed at the least cost solution. The parameter which most significantly affected the results was the magnitude of load to which the framing system is subjected. Figure 23 represents the effects of variations in the load magnitude from a minimum of 30 psf to a maximum of 80 psf in 10 psf increments. A rectangular boundary is associated with each of the curves to represent the range of solutions which are within a $0.10/sq ft cost variation of the least cost solution. As the magnitude of the load increased, the range within which no more than a $0.10 increase is possible, is reduced. Also, as the magnitude of the load increases, the least cost bay area decreases.

CONCLUSIONS

Within the constraints of a double-cantilevered girder steel joist framing system, least cost objectives can be accomplished with rectangular rather than square bays. The economic bay area falls in a range between 750 sq ft and 1,250 sq ft for conditions involving shallow spread footings. Deep foundations have the effect of increasing the size of the least cost bay area. Significant economies can be realized using a framing system in which the joist span is 1.25 to 1.75 times the girder span.