10. GA BASED OPTIMIZATION OF COMPOSITE TRUSSES

10.1 GENERAL REMARKS

Composite truss is one of the best potential solutions for beam span in the range of 12 to 30 meters [103]. Composite action of steel truss with concrete deck gives cost effective solution and voids between bracing members of steel truss facilitates service zone. **Figure 10.1** shows the schematic diagram of composite truss where in steel truss is connected with concrete slab using shear studs which make these two elements act united and give composite action. The concrete floor slab used as a part of compression chord of the truss is less vulnerable to buckling failure. Also, the concrete can more economically carry the compression, whereas it is very weak in tension. In composite truss system, thus, the relative merits of steel and concrete as construction material are fully exploited. It can be further economised by optimizing the truss depth, panel width and size of the truss members. In multi storey buildings the composite truss systems also reduce the total height of the building, by accommodating the services within the depth of the truss, thus integrating structural, mechanical and electrical systems within the floor space.

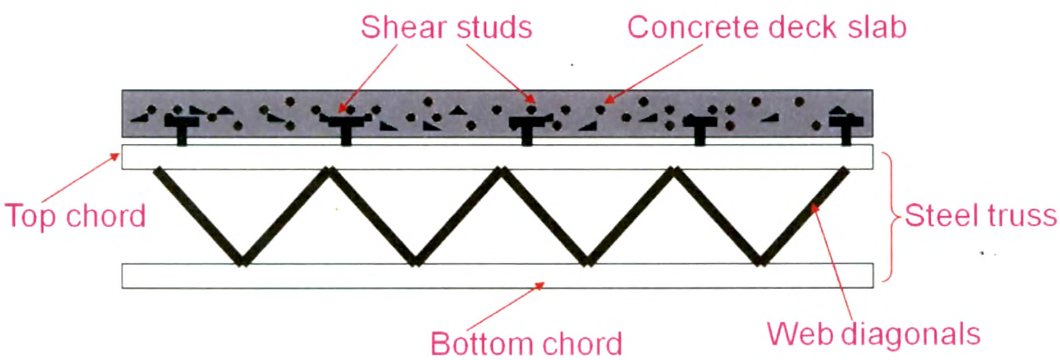

Fig. 10.1 Components of a Composite Truss [49]

Warren truss, Pratt truss and Warren truss with Vierendeel panel are most commonly used configurations in composite truss. Optimization of steel concrete composite truss floor system combines optimum design of RCC slab and optimum design of steel truss so as to get maximum benefits with minimum resources. Design of slab is governed by minimum thickness, minimum profile sheet depth and minimum concrete cover above shear studs. Further, reduction in slab thickness is not permitted in the standards. Moreover, increase in the slab thickness increases the loading on composite truss and ultimately does not help in optimization process. There are following three possibilities in optimization of a truss:

- Size optimization,
- Configuration optimization,
- Topology optimization.

In size optimization, optimums cross sectional area of truss members are calculated to minimize the cost of steel truss subjected to various functional and behavioural constraints. Configuration optimization is the combination of size and geometry optimization. In configuration optimization initial truss geometry (ground structure) is supplied to the optimization algorithm which finds optimum joint co-ordinates and optimum member criterion so as to minimize the cost and maximize the strength. In this case final optimum solution is greatly influenced by the initial ground structure supplied to the algorithm. Here, number of members and joints are not held fixed.

The most complicated and most general optimization in trusses is the topology optimization. In this category, algorithm is not supplied with initial ground structure but it is free to choose any geometry in the search space provided by the designer. Due to large number of design variables, however, topology optimization is computationally more involved and very time consuming but capable of evolving new and innovative design solutions.

From the literature survey it is clear that the amount of work done related to optimization of composite steel-concrete structures is very limited. To the best of author's knowledge no work has been reported in the literature on optimization of composite truss using GA. As varieties of shapes and sizes of composite structural components are in use in construction, it is very much desirable to find optimum composite truss parameters.

In the present work, a software is developed for the design of composite truss using Limit State Method of design. The objective is to facilitate the design of the composite truss having Warren truss, Warren truss with Vierendeel panel or Pratt truss configurations. GA based configuration optimization algorithm is developed with the objective of minimizing the cost of composite truss considering the configuration optimization parameters of the composite truss as truss depth, panel width and size of truss members. To facilitate menu driven input of data and continuous display of the improvement in configuration of steel truss during the optimization search process, Visual Basic.Net environment is selected. The Limit State Method of design consistent with BS 5950:2000 [93] is employed for the design of composite floor and truss system following the guidelines given by Mediate [51].

10.2 TRUSS CONFIGURATIONS IN COMMON USE

A large number of truss configurations may be worth considering for use as composite truss. Pratt and Warren truss configurations are most common and desired ones. Pratt Truss (**Fig. 10.2**), although theoretically the most efficient truss configuration, has limited usefulness for typical floor framing. Additional members increase fabrication costs and relatively small free area between the diagonals greatly reduces flexibility for services sizes and locations.

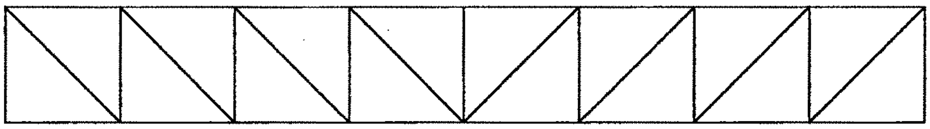

Fig. 10.2 Pratt Truss

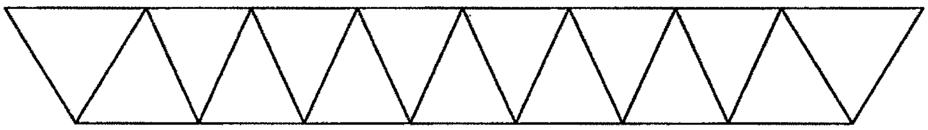

Fig. 10.3 Warren Truss

In a conventional Warren truss as shown in **Fig. 10.3**, configuration limits service duct sizes to those that will fit between the diagonal bracing members. However, the use of Vierendeel panels without bracing members is permitted in most truss applications, which greatly increases the zone of services. The Vierendeel panel should be located at the mid span so that the size of the openings is maximum and the minimum stiffening of the truss chords is required. **Figures 10.4** and **10.5** show Warren truss and Pratt truss respectively with central Vierendeel panel.

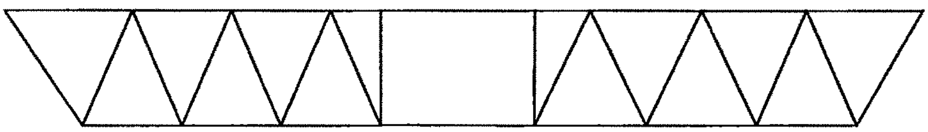

Fig. 10.4 Warren Truss with Vierendeel Panel

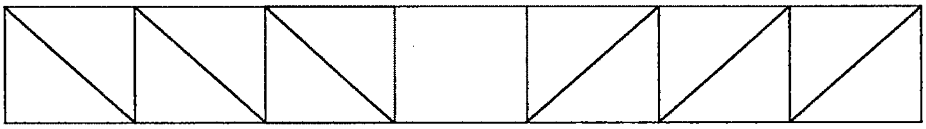

Fig. 10.5 Pratt Truss with Vierendeel Panel

The bottom chord can be either extended to the support or terminated at the last panel point as shown in **Figs. 10.2, 10.5** and **10.6.** Generally the chord can terminate before the supports where trusses are used as secondary members. If the truss acts as a primary beam, or supports heavy point loads, it is recommended that the chord extends to the support to provide improved resistance to "flange tripping". i.e. lateral buckling of the bottom chord in tension.

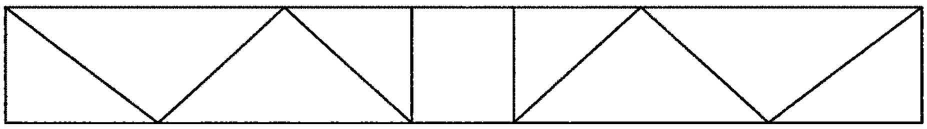

Fig. 10.6 Warren Truss with Vierendeel Panel and Bottom Chord up to Support

In order to reduce the span between the top chord nodes and hence to minimize the top chord size, vertical members can be introduced in Warren trusses as shown in **Fig. 10.7** and **10.8.** This is especially advantageous in the first panel on either side of the Vierendeel panel where top chord bending and axial forces are the highest. Same benefits can be achieved by varying the panel spacing. However by experience it has been found that the minimum size of the top chord is often governed by handling requirement. Adding vertical members or varying the panel spacing is not advantageous until spans exceed 15 m.

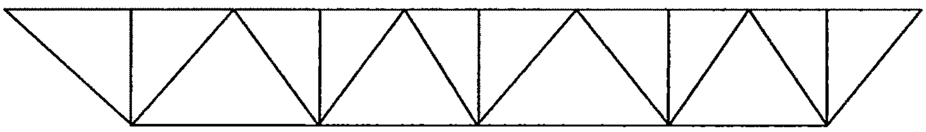

Fig. 10.7 Warren Truss with Vertical Members

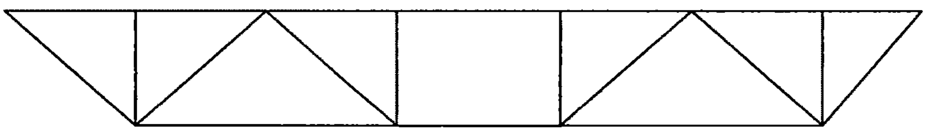

Fig 10.8 Warren Truss with Vierendeel Panel and Vertical Members

Ideally the centriodal axes of the compression and the tension web members should meet at the same 'node' point to avoid eccentricity. However, in the composite case the effective 'node' point is within the slab and therefore the chords can be 'separated' slightly. If the chord members are rectangular hollow sections (RHS), the bottom chord web joint can accommodate a slight eccentricity. However, for tee section chords, the additional moments that are induced can influence the required section size. Therefore, for most of the applications, concentric joints should be used, with the connection centerlines of the web members separated only as necessary to simplify welding [51].

10.3 Analysis and Design of Composite Truss

10.3.1 Configuration

It is very important to judiciously select initial parameters for the truss. Since the supports are considered as simply supported, the span to depth ratio should be typically between 15 and 20. In addition to the depth of the truss, there should be adequate depth allowance (150 – 200 mm) below the bottom chord to take care of deflection, lighting, ceiling system and fire protection.

In order to maximize the clear zone through the truss, the slope of the bracing diagonals should preferably be 45° or less with the horizontal. The most efficient proportion has been found to be having panel width to truss depth ratio of 3:1 considering a slope of around 30° as shown in **Fig. 10.9.** Though larger panel size leads to slightly heavier section due to higher load, less number of members and lower fire proofing costs are the gains. The Vierendeel panel size should be chosen such that it can accommodate the major service duct and the panel width opening should not exceed two times the depth of the truss.

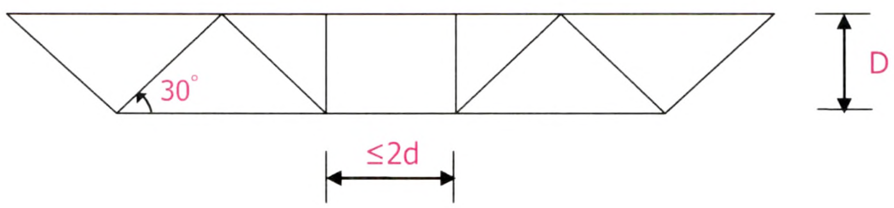

Fig. 10.9 Preferred Dimensional Stipulation for Composite Truss

The top chord section should take into account the following:

- Ability to span between the braced nodes and supporting the load during construction.
- Stability during erection process and to provide bearing support for the profiled decking. A minimum width of 120 mm is usually acceptable.
- For through deck welding cases, the minimum flange thickness to be 8 mm.
- Adequate depth for welding of the bracing members.
- Resistance to local bending at Vierendeel panels.

The effective length of all the members is often based on the assumption that the ends are pinned. Whilst this may be true of the steel truss, the bracing members in a composite truss achieve partial restraint from the slab. A reduced effective length (say 15% reduction to 0.85L) may be considered in such cases.

10.3.2 LOADING

Primary load cases applicable to this kind of truss are listed below:

- Self weight of the truss and concrete slab.
- Dead load due to service loads, raised floor system and ceiling.
- Construction load consisting of the imposed loading on the deck prior to development of composite action of the truss.
- Imposed loading consisting of the design floor loading and the partitions in accordance with BS 6399: Part 1.

Typical unfactored values that can be considered are:

Construction load = 0.5 kN/m^2 or point load = 4 kN

Imposed load = 4.5 kN/m^2 including partitions

Imposed loading of 4.5 kN/m^2 on the floor is the best compromise taking into consideration a minimum of 3.5 kN/m^2 UDL, stipulated for commercial buildings and the requirements to allow future flexibility in the floor usage. The value considered can usually accommodate all the potential office loadings. The longer the span, the lesser is the probability that a given total load will be attained. Hence long span structures can be designed for lower imposed load than short span structures.

For non composite case, self weight and construction loads are considered. After the concrete has been cast, all the primary loads are to be considered, excluding the construction load. Unbalanced loading along the span leads to larger shear forces in the members near the mid span than the uniform load case. While considering construction conditions, the steel truss is designed to support the weight of wet concrete and a construction load equivalent to those mentioned above. The minimum size of the top chord section may be influenced by local bending. The moment capacity of the section may be determined by referring to **Fig. 10.10.**

$$M_c = R_t(D_t - X_b - X_t) \qquad \text{...(10.1)}$$

where, R_c = compressive resistance of the top chord, D_t = overall depth of the truss, X_c = elastic centroid depth of top chord from top of truss, and X_b = elastic centroid depth of bottom chord from bottom of truss.

This moment capacity should exceed the factored moment calculated using the load factors given in BS 5950 [93]. The chords may be checked for the combination of tension or compression and local moment. Each chord may be assumed to resist a shear force in proportional to its stiffness and the local moments is equivalent to the shear force times half the opening width.

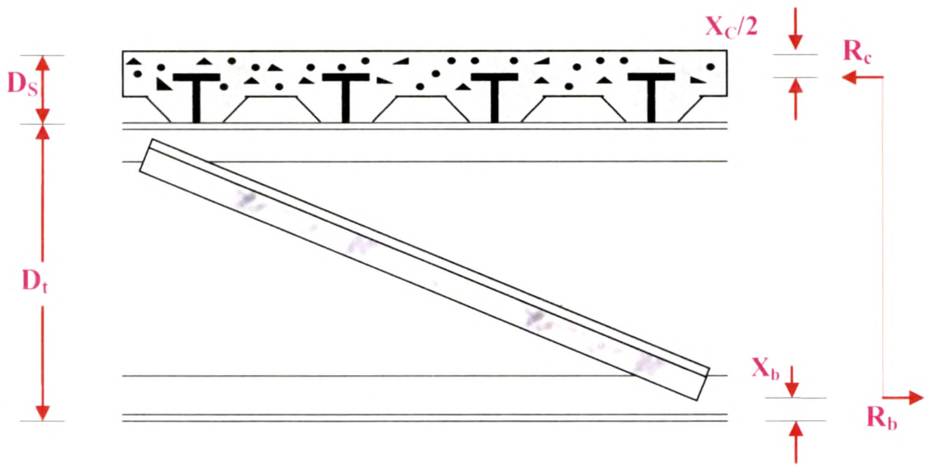

Fig. 10.10 Moment Capacity of Composite Truss

10.3.3 Moment Capacity

In limit state method total factored moment applied to the beam should be less than the moment capacity of the composite truss. For construction condition additional checks are required for the steel truss. This is usually important when considering the design of the top chord and Vierendeel panel. The moment capacity of the steel truss system at the point of maximum moment is determined by compression in the top chord and tension in the bottom chord.

In composite truss, the compression force may be considered to be resisted by the concrete or composite slab with a consequent increase in the lever arm from the bottom chord to the point of compression in the slab. The important parameters in this analysis are compressive resistance of the concrete slab, Rc and the tensile resistance of the bottom chord, R_b. The contribution of the top steel chord is ignored because of concern about the amount of strain in the bottom chord necessary before the full tensile action of the top chord is developed. The resistance of the bottom chord is given by:

$$R_c = 0.45 f_{cu} * B_c (D_s - D_p) \quad \text{... (10.2)}$$

where, f_{cu} = cube strength of concrete, B_c = effective breadth of the concrete slab, D_s = overall slab depth, and D_p = depth of the deck profile.

B_c is defined as the sum of the effective breadths (BS: 5950 Part 3) as be of the portion of flange on each side of the centerline of the steel beam. For slabs spanning perpendicular to beam, b_e should be equal to Lz / 8 but not greater than b, and slabs spanning parallel to beam, b_e should be equal to Lz/8 but should not be greater than 0.8b. Lz is the distance between points of zero moment. For simply supported span Lz is equal to effective span L.

$$R_b = A_b * P_y \quad \text{... (10.3)}$$

where, A_b = cross sectional area of the bottom chord, and P_y = design strength of steel.
In most cases $R_b < R_c$ and so the moment capacity M_c of the composite truss (**Fig. 10.10**) is given by the tensile action of the bottom chord multiplied by the lever arm to the point of compression in the slab. Hence,

$$M_c = R_b(D_t + D_s - 0.5X_c - X_b) \quad \text{... (10.4)}$$

where, $X_c = (D_s - D_p)R_b/R_c$, d = depth of composite truss, D_t = overall depth of steel truss given as, $(d + X_t + X_b)$, and X_b = depth of elastic centroid of the bottom chord of the truss.

The increase in moment capacity of a composite truss in comparison to steel truss is usually between 20% to 30%. Other benefits of composite action are transfer of local moment at Vierendeel openings and substantial increase in the overall stiffness of the truss.

10.3.4 Shear Capacity And Longitudinal Shear

The shear capacity of the truss can be evaluated from first principles by considering the component forces in the bracing members. All connections are assumed to be pinned. For the conventional 'Warren' truss the inner bracing members are in compression and the outer members are in tension. If the bracing members are positioned at an angle equal to θ with the horizontal their maximum tensile force is:

$$F_t = Reaction/\sin\theta \quad \text{... (10.5)}$$

This tensile force is counteracted by a compression force in the next bracing member remote from the support. The tension and compression forces are equal if vertical member is not provided. If vertical member is provided, the compression force is reduced because the local force is transferred from the slab via the member.

An additional requirement is that sufficient transverse reinforcement is placed in the slab so as to permit a smooth transfer of force from the shear connection into the concrete. The requirements for transverse reinforcement are given in Clause 5.6 of BS: 5950 Part 3.

10.3.5 MEMBER CHECK

Chord Members

The sizes of the individual members are checked according to the forces and moments for each combination of load cases. The output of the different load combinations is used to determine the most adverse load case to be used for design at the ultimate limit State. In the composite condition the truss model analysis leads to axial forces and moments in the top chord member and concrete slab separately. The most severe combination of axial compression and bending moment determines the size of the top chord member. The bottom chord will primarily be subjected to tension. The effects of the combined moment and axial load must be taken into account for all the load cases to determine the size of the bottom chord.

The direction of buckling will determine the effective length of the chords. For out of plane buckling, the effective length is conservatively taken as distance between the nodes. For in plane buckling, the effective length may be taken as 0.85 times the distance between the node points representing the influence of partial restraint of the nodes by the bracing members. The top chord is fully restrained except for in plane buckling at the construction stage.

Bracing Members

A compression force is developed in the steel top chord due to the component of force transferred via the bracing members. However, in a composite truss this force is then transferred uniformly to the slab by the shear connectors attached to the top chord. The top chord must therefore be able to resist the local forces at the nodes even though its effect on overall bending is neglected. The bracing forces may be calculated at all points along the truss by considering equilibrium at the nodes. The size of the bracing members may be reduced, if desired, towards the lower shear zones.

Local moments at the braced nodes in the bottom chord are usually ignored at this stage. Where there is an eccentricity in the projected centroid of the members, the eccentric force gives rise to an overall bending effect at the node, which is resisted by the members broadly in effects as mentioned in BS 5950 Part 1. For nodes in the top chord connected to the composite slab, the bracing members may be separated slightly as their projected centroid need not align with the steel top chord but with the center of the concrete slab. Any further eccentricity may be treated as a local moment in the slab.

10.3.6 DEFLECTION CHECKS

The second moment of area I_c of a composite truss may be evaluated by converting the concrete area to an equivalent steel area. Thus the composite truss becomes equivalent to two concentrated blocks of steel area separated by the distance between the mid depth of the slab and the bottom chord. This leads to the following expression for composite stage:

$$I_c = \frac{A_b A_c/m}{\left(\frac{A_b + A_c}{m}\right)}\left[D_t + \frac{D_s + D_p}{2} - X_b\right]^2 \qquad \ldots(10.6)$$

Where, A_c = cross sectional area of concrete in the effective breadth of slab = $(D_s - D_p) \times B$, m = modular ratio and A_b = cross sectional area of the bottom chord.

The imposed load deflection of a composite truss subjected to uniform load of w/unit length may therefore be calculated from the assumption that bending effect is dominant. Hence:

$$\delta_c = \frac{5 * w_i * l^3}{384 * E * I_c} \qquad \ldots(10.7)$$

where, L = truss span, E = elastic modulus of steel, I_c = moment of inertia given by Eqn. 10.6.

For long span trusses (span/depth ratio > 15), this consideration might work well but an additional component of deflection due to axial strain in the bracing members may need to be considered in deep trusses or those subjected to heavy point loads. A 10 % allowance for these additional deflections is usually appropriate for the truss proportions normally used. The deflection for imposed load should be limited to span / 360.

Except for calculation of self weight any further checks on serviceability performance are not needed. The second moment of area of the steel truss I_t for pre composite stage is obtained by considering the separation of the bottom chord and the top chord of area A_t. Thus,

$$I_t = \frac{A_b A_t}{A_b + A_t}[D_t - X_b - X_t]^2 \qquad \ldots(10.8)$$

Generally, it is found that I_t is smaller than I_c suggesting that self weight deflection may be of similar magnitude to the imposed load deflections. The deflection in pre composite stage is given as,

$$\delta_t = \frac{5 * w_i * l^3}{384 * E * I_t} \qquad \text{...(10.9)}$$

Hence total deflection is given by $\delta_c + \delta_t$.

Deflection checks can be carried out in two stages:

- Deflection due to self weight and construction loads on the non composite truss structures.
- Deflection due to dead and imposed loads on the composite truss structures.

The elastic modulus of concrete used for the dead and imposed load conditions should be consistent with the duration of loading under consideration. The truss is considered to be a simply supported member and the second moment of area of the steel truss is calculated from the properties of the top and bottom chords (and slab in the composite case) only. Deflections calculated in this manner does not take into account the shear deformation of the bracing members and are therefore about 15% less than the values obtained from the analysis. The result should be checked against the following limitation.

$$\delta = Span/325 \qquad \text{...(10.10)}$$

10.4 Configuration Optimization Problem Formulation

The problem of configuration optimization of composite truss can be formulated as:

Find, (x)

To minimize, $C_T = C_S + C_C$

Subject to, $g_i(x) < 0$ ... (10.11)

where, $C_T(x)$ is the total cost of composite truss, C_S is the cost of steel truss, C_C is the cost of concrete slab, x is the vector of design variables and $g_i(x)$ is the i^{th} constraint function.

10.4.1 Design Variables

Following design variables are considered which not only define the complete geometry of the truss but also the member section properties.

- **Depth of truss:** The cost of the truss depends on the length of the members which is directly affected by the depth of truss. Depth of simply supported composite truss is considered to vary from span/15 to span/20.
- **Number of panels:** For a given truss depth and span, number of panels decides the angle of bracing member. A slope of about 30°, creating a panel width to truss depth ratio of 3:1 has been found to be most efficient proportion.Number of panels are selected here to keep bracing member inclination in the specified range (30° to 45°).
- **Member cross-section properties:** Properties of the sections suitable to four groups of truss members (viz. Top-chord (TC), Bottom-chord (BC), Diagonal Tension (T) and Diagonal Compression (C)) are stored in a common database with a unique integer number assigned to each section. These four binary design variables are decoded into integer numbers corresponding to which, section properties are extracted from the database and used in the design. These variables takes only discrete values corresponding to rolled steel sections stored in the database which makes the optimum solution practically feasible.

10.4.2 DESIGN CONSTRAINTS

Safety is of prime importance in any structural design. Thus, while optimizing any structural component there should be no compromise with safety. This requires fulfilment of certain condition and constraints, violation of which would make the structure unsafe. In structural problems, constraints are formed by setting relationship between function of design variables with the resource values such as permissible stress, permissible deflection etc. Thus, constraints in the optimization process prevent the search to enter the infeasible region. The following constraints are imposed here.

- **Moment constraint:** In limit state design the moment capacity of the composite truss (M_c) should exceed the total factored applied moment [104]. The constraint can be written as:

  $$M_f < M_c \qquad \dots (10.12)$$

  Corresponding function for this constraint is;

  $$g_o(x) = Max\,(M_f / M_c - 1,\, 0) \qquad \dots (10.13)$$

- **Member force constraint:** This constraint ensures that the capacity of the truss member is more than the actual load induced in the member. The constraints for top chord, bottom chord, diagonal tension and diagonal compression members are:

$$TC_f < TC_c,\ BC_f < BC_c,\ T_f < T_c,\ C_f < C_c \quad \text{... (10.14)}$$

where, TC_f, BC_f, T_f, C_f and TC_c, BC_c, T_c, C_c are factored loads and capacities of top chord, bottom chord, diagonal tension and diagonal compression members respectively. The associated constraint functions are:

$$g_2(x) = Max\,(TC_f / TC_c - 1,\ 0)$$
$$g_3(x) = Max\,(BC_f / BC_c - 1,\ 0)$$
$$g_4(x) = Max\,(T_f / T_c - 1,\ 0)$$
$$g_5(x) = Max\,(C_f / C_c - 1,\ 0) \quad \text{... (10.15)}$$

- **Deflection constraint:** This constraint checks the serviceability limit state and is given by

$$\delta_{max} < \delta_{permissible} \quad \text{... (10.16)}$$

where, δ_{max} is the maximum deflection in the composite truss and $\delta_{permissible}$ is permissible deflection given by span/325.

10.5 GA IMPLEMENTATION

GA involves initial random selection of solutions from the available search space as defined by the upper and lower bounds of the design variables. This initial population of solutions is generated in the binary form. The solution in GA is the series of design variables defining complete solution which in case of composite truss optimization represents geometry of truss and cross-section properties of the truss members. Usually number of variables representing member section properties equals to the number of truss members. However, in the present study, the variables are reduced to four by grouping similar type of the members in four groups. In addition there are two more variables that define configuration of truss of a selected geometry. These variables are depth of truss and number of panels. Thus one GA solution string, consisting of six binary substrings, defines the six design variables.

The binary string representation scheme is used for all the variables. The accuracy of the variable depends on the number of bits in each string. Each potential solution is represented by a single binary string called the main string, which is then divided into six smaller strings each representing a design variable listed above. The binary strings are then converted into their decimal equivalents and are mapped between upper and lower bounds to obtain the values of the variables. The mapped values shown in **Fig. 10.11** represent the configuration as shown in **Fig. 10.12.**

Initial population of randomly selected solution strings in binary form is decoded to find actual values of design variables and corresponding structures are generated. The generated truss structures are analyzed to find constraint functions $g_i(x)$ and objective function $C_T(x)$ which is multiplied by penalty function $(1 + kC)$ to find the penalized objective function C_p given by

$$C_p = (1 + kC)C_T(x) \qquad \dots (10.17)$$

where k is penalty parameter which is adopted as 10 in the present study and C is cumulative constraint function.

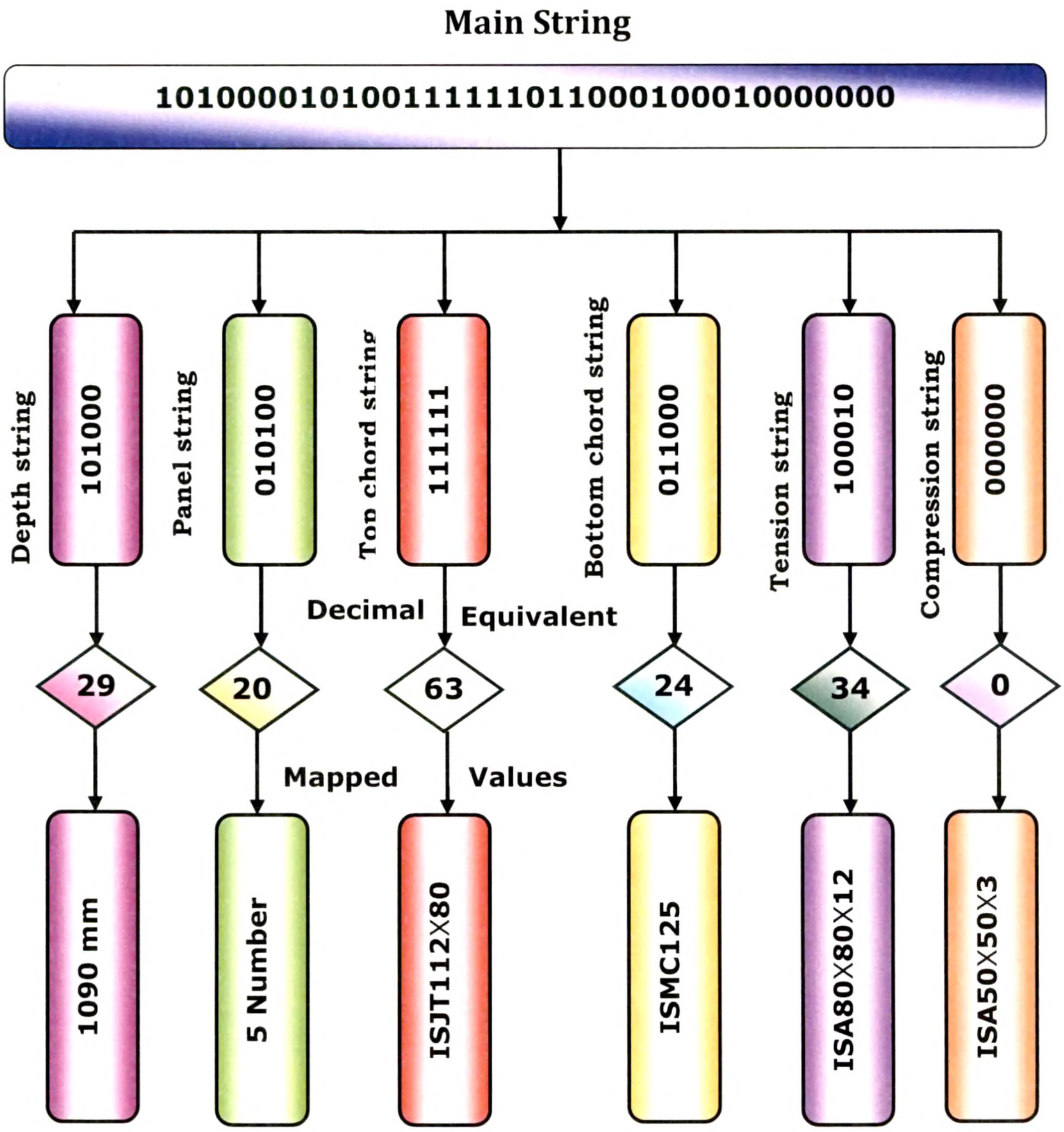

Fig. 10.11 Representation of Solution String in Graphical Form

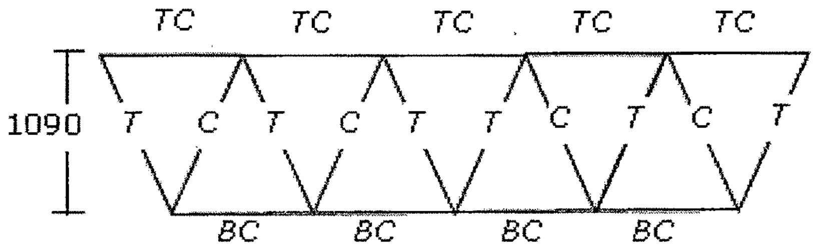

Fig. 10.12 Corresponding Structural Representation of Solution String

From the above equation it is clear that for a feasible solution all the constraint functions will be zero and objective function will not be penalized. For infeasible solution (1 + kC) value will be more than 1 which will increase the penalized cost to be minimized and decrease the fitness function formulated as [10]:

$$F(x) = 1/(1 + C_p) \qquad \text{... (10.18)}$$

The fitness function is the measure of goodness of a solution in the optimization. After calculation of fitness for each solution of the initial population (1^{st} Generation), a new population (2^{nd} Generation) is produced by applying GA operators such as selection, crossover, mutation and elitism. This new population is further evaluated to calculate the fitness values and again the GA operators are employed to produce 3^{rd} generation. The process is repeated until stopping criteria is satisfied. The solution having maximum fitness among all generations is considered as the optimum solution.

10.6 PROGRAM DEVELOPED FOR THE COMPOSITE TRUSSES

The GA based inbuilt functions like "Rnd function", "Mid, Left and Right function" etc. are used in the program which is developed in VB.NET environment. A modular approach is adopted by developing a number of subroutines such as Subroutine *Genetic* to develop strings for given variable of design. Subroutine *Fitness Scaling* is written to scale the fitness. Subroutine *Crossover_Mutation* is made to perform Crossover and Mutation. *Fitness* function is added to calculate fitness based on the objective function which is the total cost of the composite truss in present work. A number of menus and forms are developed to facilitate pre- and post- processing of data.

The procedure to obtain the solution is illustrated here with the help of the screen shots of the forms given in **Figs. 10.13** to **10.22.**

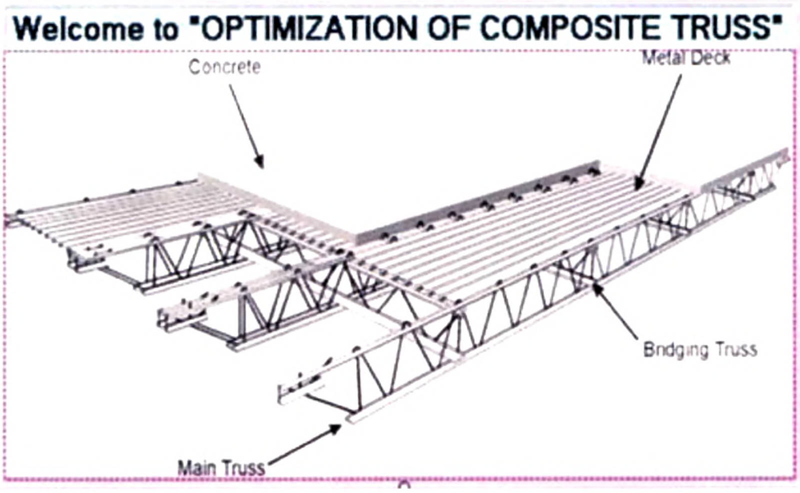

Fig. 10.13 Start up Screen for Composite Truss

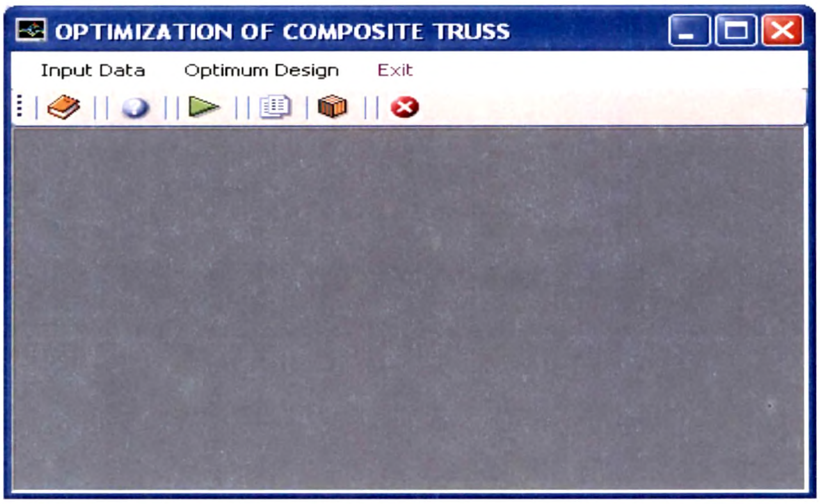

Fig. 10.14 Form for Option Menus and Tools

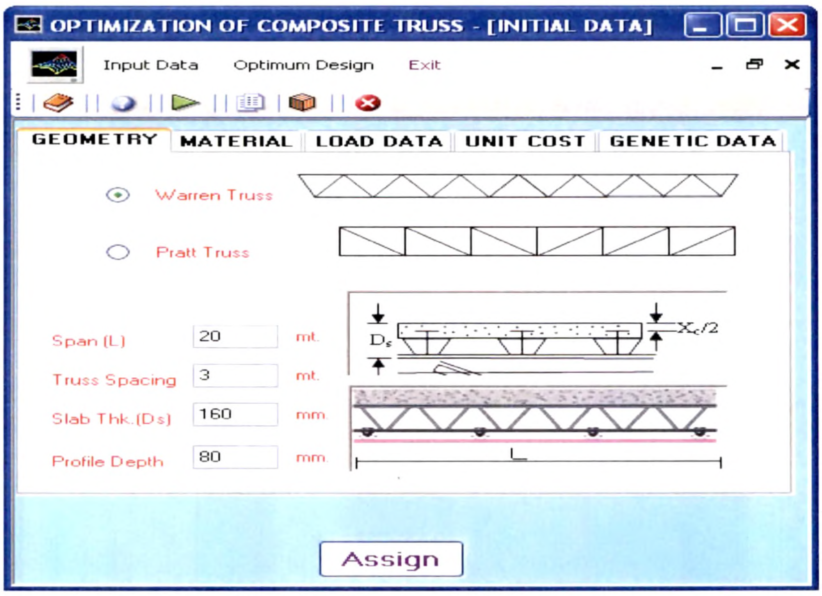

Fig. 10.15 Form for Entering the Geometry Data

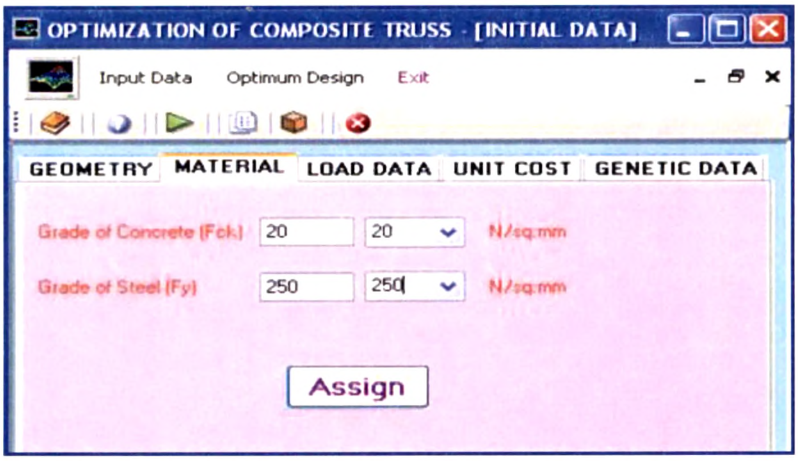

Fig. 10.16 Form to Enter Material Data Related to Concrete and Steel

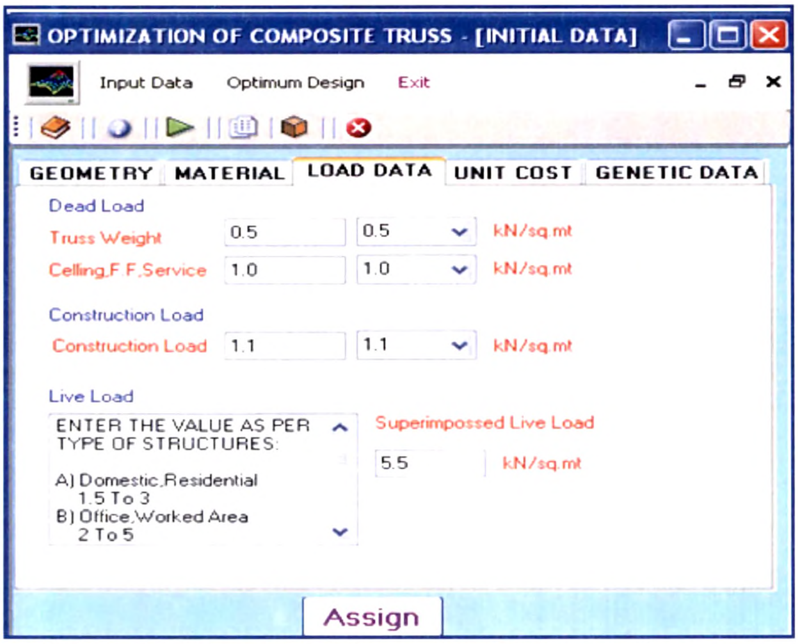

Fig. 10.17 Data Form for Dead, Live and Construction Loads

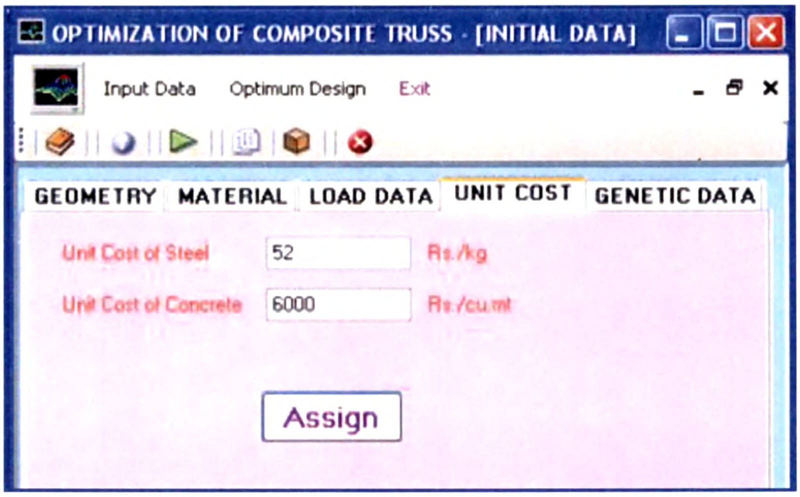

Fig. 10.18 Form for Entering Unit Cost of Material

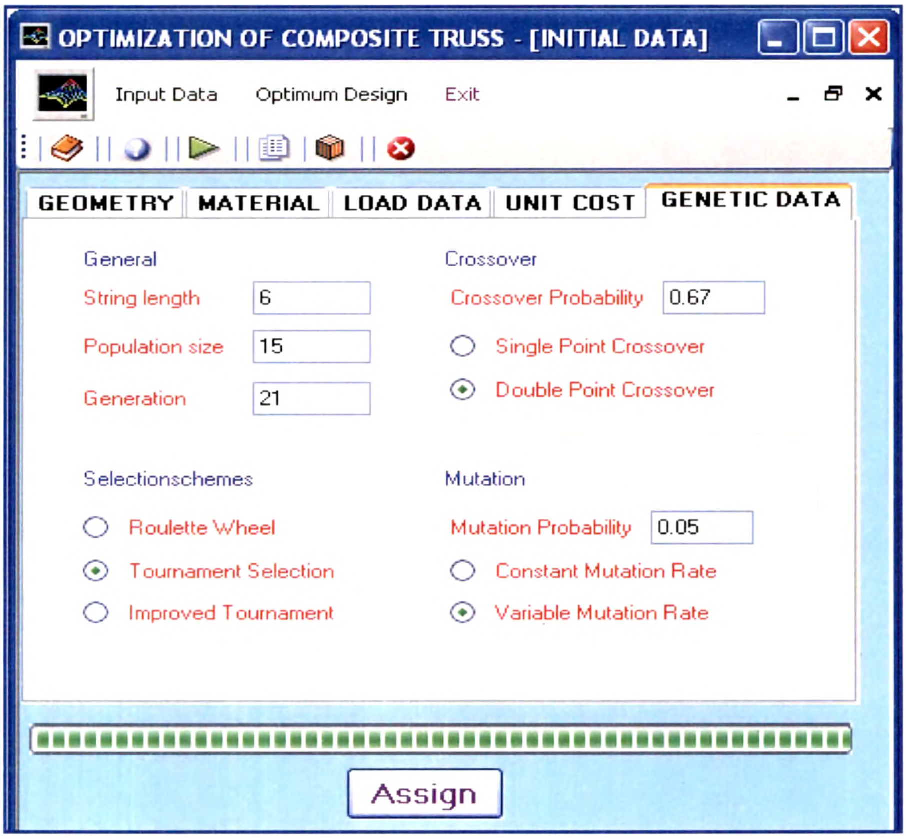

Fig. 10.19 Menus for Supply of Genetic Data

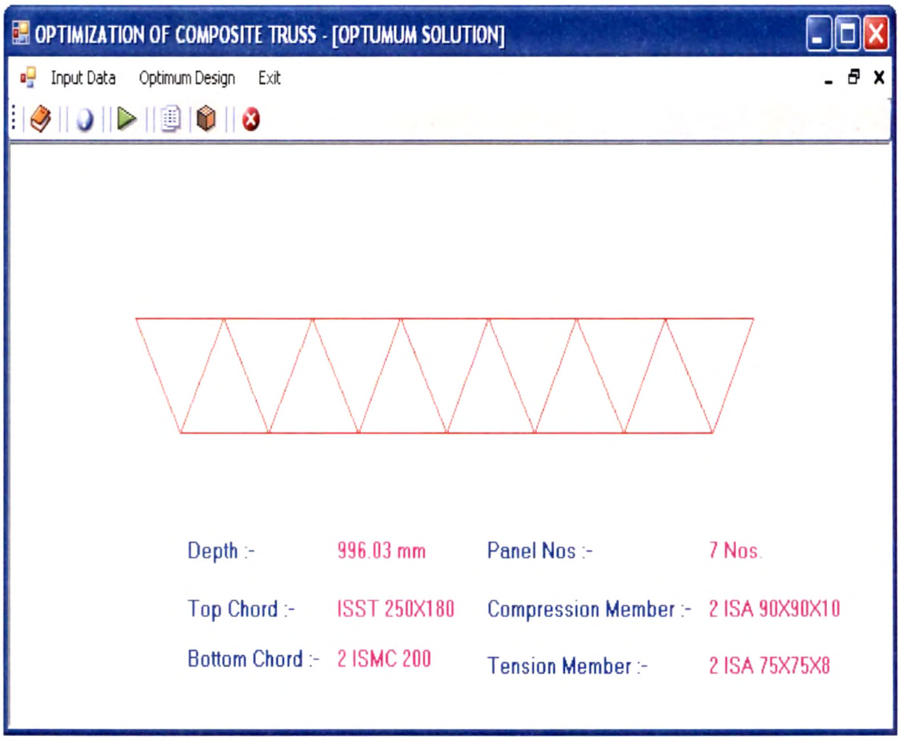

Fig. 10.20 Perform Optimum Design_1

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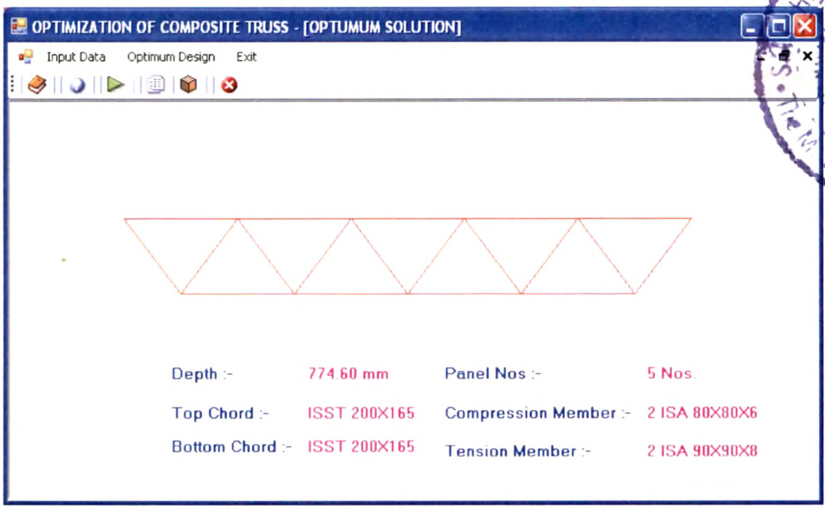

Fig. 10.21 Perform Optimum Design_2

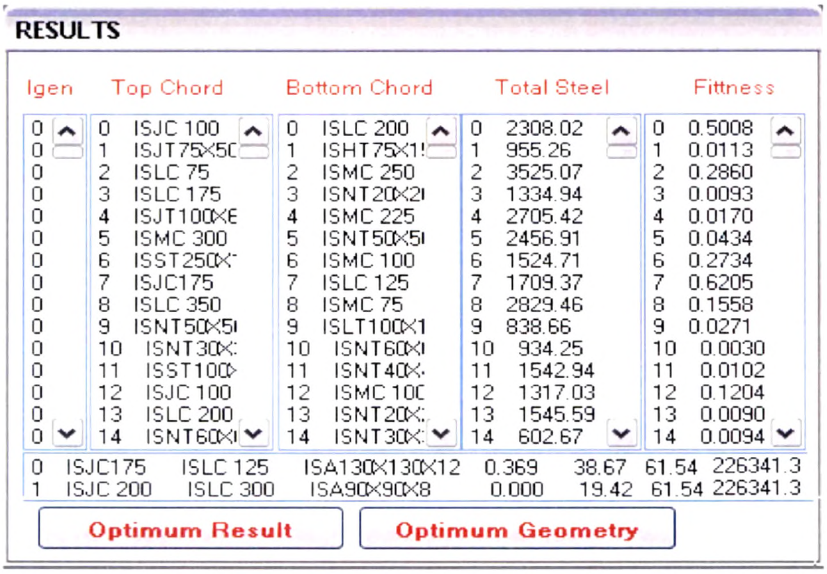

Fig. 10.22 View Generation History

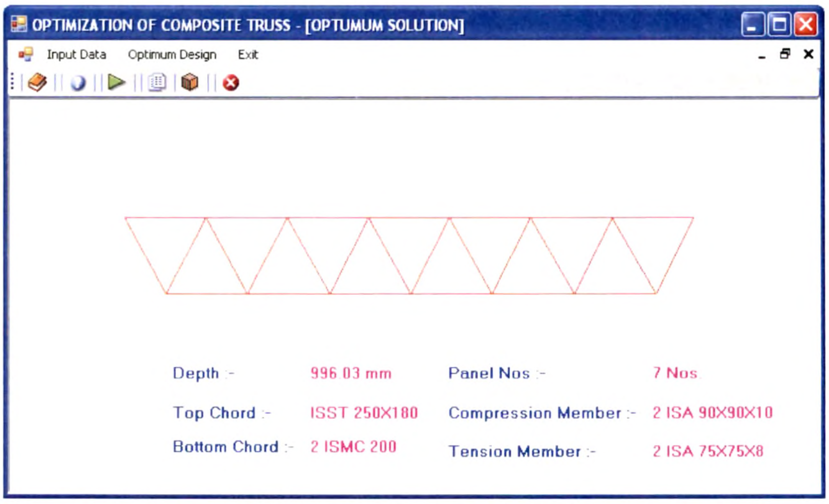

Fig. 10.23 Display of Optimum Geometry on Screen

10.7 Optimum Design Example of a Warren Truss

➢ **Geometry data**

- Truss type = Warren truss
- Span = 15 m
- Truss spacing = 3.5 m
- Slab thickness = 160 mm
- Profile depth = 80 mm

➢ **Material data**

- Grade of concrete = M20
- Grade of steel = Fe 250

➢ **Load data**

- Load due to ceiling, F.F. and Services = 1.1 kN/sq. m
- Construction load = 1 kN / sq. m
- Superimposed live load = 5.5 kN/sq. m.

➢ **Unit cost data**

- Unit cost of steel = 52 ₹/ kg.
- Unit cost of concrete = 6000 ₹/ cum.

➢ **Genetic data**

- String Length – 6
- Population Size – 15
- Generation – 21
- Type of Crossover – Double Point Crossover
- Crossover Probability – 0.67
- Selection Scheme – Roulette Wheel Scheme
- Mutation Probability – 0.05 with variable mutation rate.

Output

The optimum solution produced by the software is shown **Fig. 10.24.**

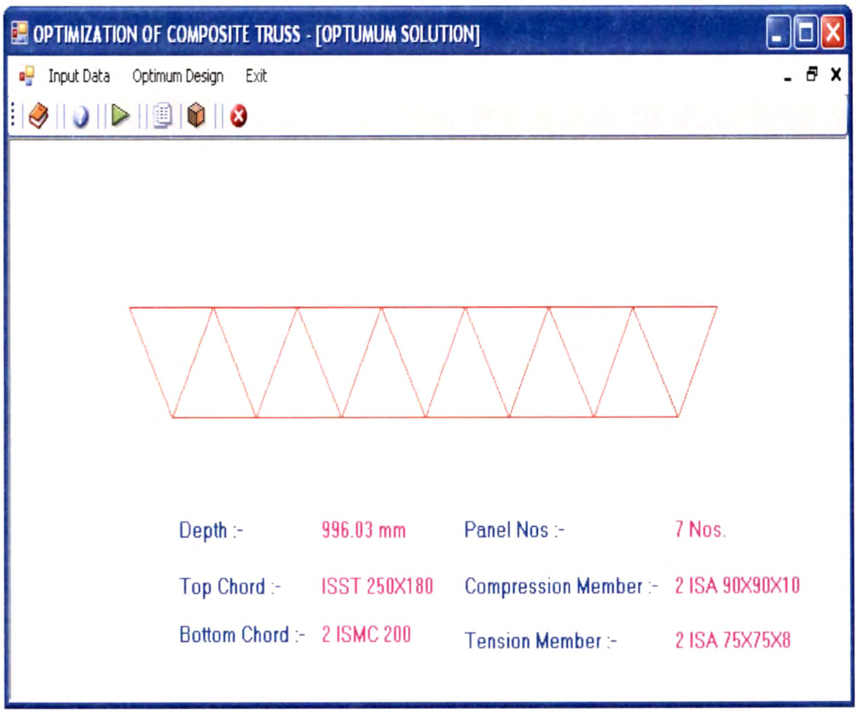

Fig. 10.24 Optimum Solution for Warren Truss (15 M Span)

10.8 Optimum Design Example of a Pratt Truss

Design of deck type of Pratt Truss with light weight concrete slab is tried now with the following data.

- **Geometry data**
  - Truss type = Pratt truss
  - Span = 15 m
  - Truss spacing = 3.5 m
  - Slab thickness = 160 mm
  - Profile depth = 80 mm
- **Material data**
  - Grade of concrete = M20
  - Grade of steel = Fe 250
- **Load data**
  - Load due to ceiling, F.F. and Services = 1.1 kN/sq. m
  - Construction load = 1 kN/sq. m
  - Superimposed live load = 5.5 kN/sq. m.
- **Unit cost data**
  - Unit cost of steel = 52 ₹/ kg.
  - Unit cost of concrete = 6000 ₹/ cum
- **Genetic data**
  - String Length – 6
  - Population Size – 20
  - Generation – 23
  - Type of Crossover – Double Point Crossover
  - Crossover Probability – 0.81
  - Selection Scheme – Roulette Wheel Scheme
  - Mutation Probability – 0.07 with variable mutation rate.

Output

The final solution obtained by GA based optimization software is shown in **Fig. 10.25**

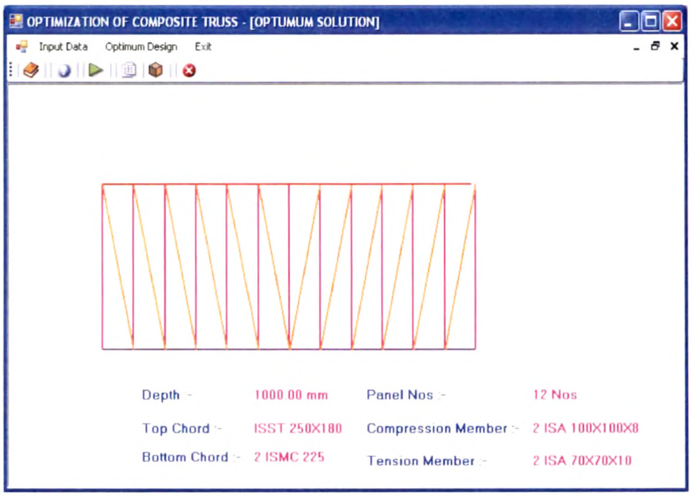

Fig. 10.25 Optimum Solution for a Pratt Truss (15 m Span)

10.9 Warren Truss Example With Vierendeel Panel

Design of a deck type Vierendeel Panel Truss with light weight concrete slab is attempted now with the following input data:

➢ **Geometry data**

- Span = 18 m
- Truss spacing = 3 m
- Slab thickness = 150 mm
- Profile depth = 75 mm

➢ **Material data**

- Grade of concrete = M20
- Grade of steel = Fe 250

➢ **Load data**

- Load due to ceiling, F.F. and Services = 1 kN/sq. m
- Construction load = 1 kN / sq. m
- Superimposed live load = 5 kN/sq. m.

➢ **Unit cost data**

- Unit cost of steel =52 ₹/ kg.
- Unit cost of concrete = 6000 ₹/cum.

➢ **Genetic Data**

- String Length – 6
- Population Size – 14
- Generation –18
- Type of Cross-over – Double Point Crossover
- Cross-over Probability – 0.87
- Selection Scheme – Roulette Wheel Scheme
- Mutation Probability – 0.06 with Variable mutation rate.

Output

The optimum solution with optimum parameters for 18 m span is depicted in **Fig. 10.26.**

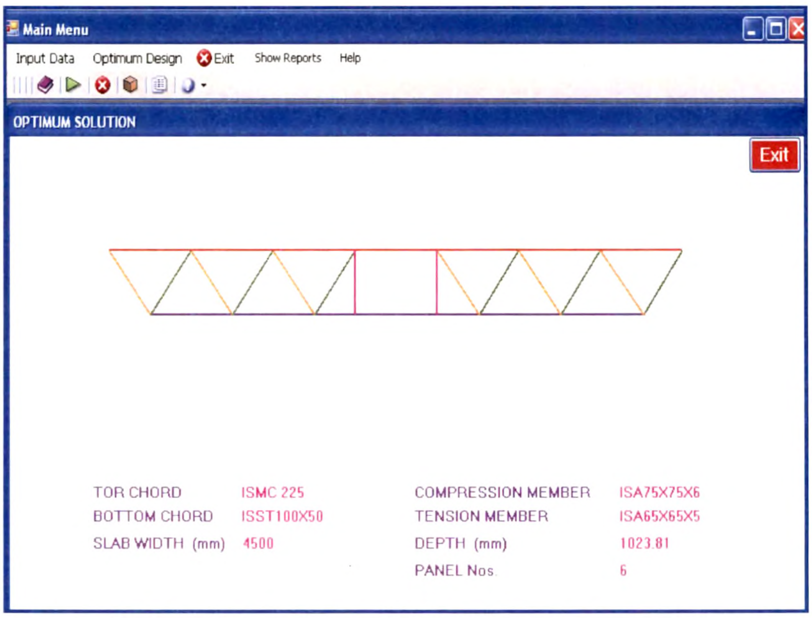

Fig. 10.26 Optimum Solution for a Warren Truss with Vierendeel Truss

10.10 COMPARISON OF RESULTS

Total four problems with 10 m, 12 m, 15 m and 20 m spans are solved using Warren truss and Pratt truss geometries for the input data given in **Table 10.1**. Results obtained using optimization software are summarized in **Table 10.2**, whereas **Table 10.3** summarizes the solution obtained by conventional method i.e. without using GA.

Table 10.1 Input Data for Warren and Pratt Truss Configurations

Span (m)	Truss Spacing (m)	Slab Thk. (mm)	Profile Depth (mm)	Steel / Concrete Grade (N/mm^2)	Total D.L. (kN/m^2)	Construction Load (kN/m^2)	Live Load (kN/m^2)
10	3	150	75	250/20	1	1	5
12	3	150	75	250/20	1	1	5
15	3.5	160	80	250/20	1.1	1	5.5
20	3	160	80	250/20	1.1	1.2	5.5

Table 10.2 Results for Warren and Pratt Truss (With GA)

Span (m)	Top Chord	Bottom Chord	Tension Member (2 ISA)	Compression Member (2 ISA)	Depth (mm.)	No. of Panels	Weight (kg)
			PRATT TRUSS				
10	ISNT100	ISNT150	55 X 55 X 10	55 X 55 X 8	700.00	10	686.70
12	2 ISMC100	ISST200	60 X 60 X 10	55 X 55 X 6	771.40	12	943.60
15	ISST250	2 ISMC225	70 X 70 X 10	100 X 100 X 8	1000.00	12	2097.10
20	ISHT150	2 ISMC350	100 X 100 X 10	100 X 100 X 10	1321.80	10	3505.70
			WARREN TRUSS				
10	ISNT150	ISHT150	55 X 55 X 8	55 X 55 X 10	500.00	6	695.40
12	2 ISMC100	ISST250	80 X 80 X 6	90 X 90 X 6	784.10	6	963.00
15	ISST250	2 ISMC200	75 X 75 X 8	90 X 90 X 10	996.00	7	1775.70
20	2 ISMC150	2 ISLC300	70 X 70 X 8	150 X 150 X 10	1100.00	8	2931.90

Table 10.3 Results for Warren and Pratt Truss (Without GA)

Span (m)	Top Chord	Bottom Chord	Tension Member (2 ISA)	Compression Member (2 ISA)	Depth (mm)	No. of panels	Weight (kg)
			PRATT TRUSS				
10	ISNT150	ISHT150	55 X 55 X 10	55 X 55 X 6	500.00	14	770.90
12	ISNT150	ISST250	60 X 60 X 10	55 X 55 X 10	700.00	12	1171.90
15	ISST250	2 ISLC250	80 X 80 X 12	70 X 70 X 10	750.00	14	2150.90
20	2 ISLC200	2 ISLC350	90 X 90 X 12	80 X 80 X 10	1000.00	14	3590.30

WARREN TRUSS							
10	ISNT150	ISHT150	70 X 70 X 6	80 X 80 X 6	500.00	7	704.00
12	ISHT125	ISST250	55 X 55 X 8	65 X 65 X 10	600.00	6	1030.70
15	2 ISLC175	2 ISLC250	70 X 70 X 10	80 X 80 X 12	750.00	7	1870.20
20	2 ISLC200	2 ISLC 350	80 X 80 X 10	80 X 80 X 12	1000.00	9	3176.90

Fig. 10.27 shows a graph between the generation number and fitness value for the composite truss of 15 m span. From the graph it is clear that the final result is obtained in the generation 31 and after that there is no further improvement. The convergence towards minimum weight is displayed in graphical form in **Fig. 10.28**.

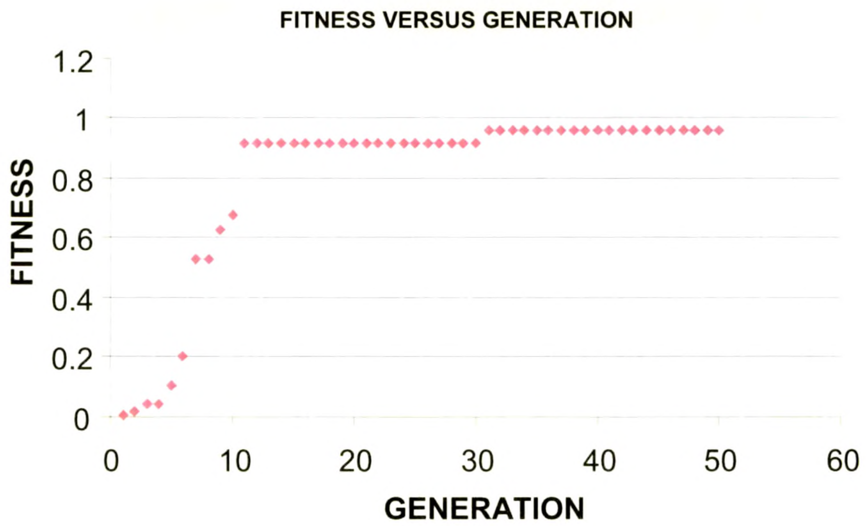

Fig. 10.27 Generation History of a 15 m Span Composite Truss

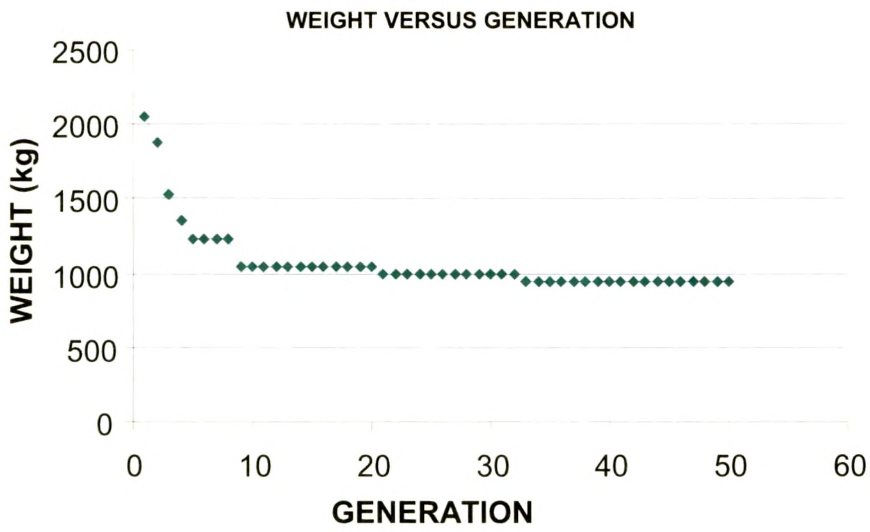

Fig. 10.28 Weight Versus Generation History

Figures 10.29 and **10.30** show results of the comparative study carried out for Warren and Pratt truss respectively. There is a noticeable reduction in the weight of the warren truss when Genetic Algorithm is used.

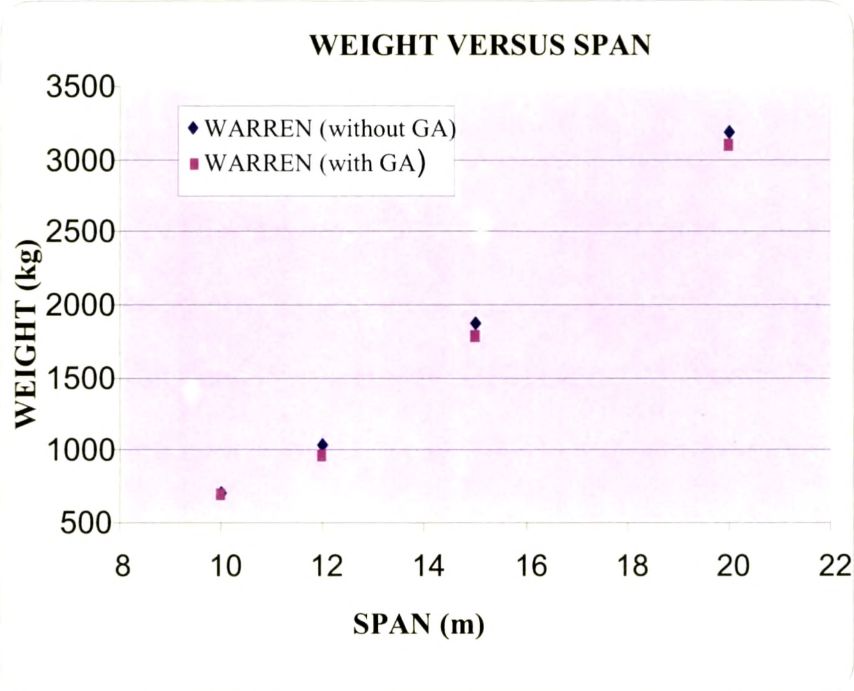

Fig. 10.29 Weight Versus Span Graph for Warren Truss

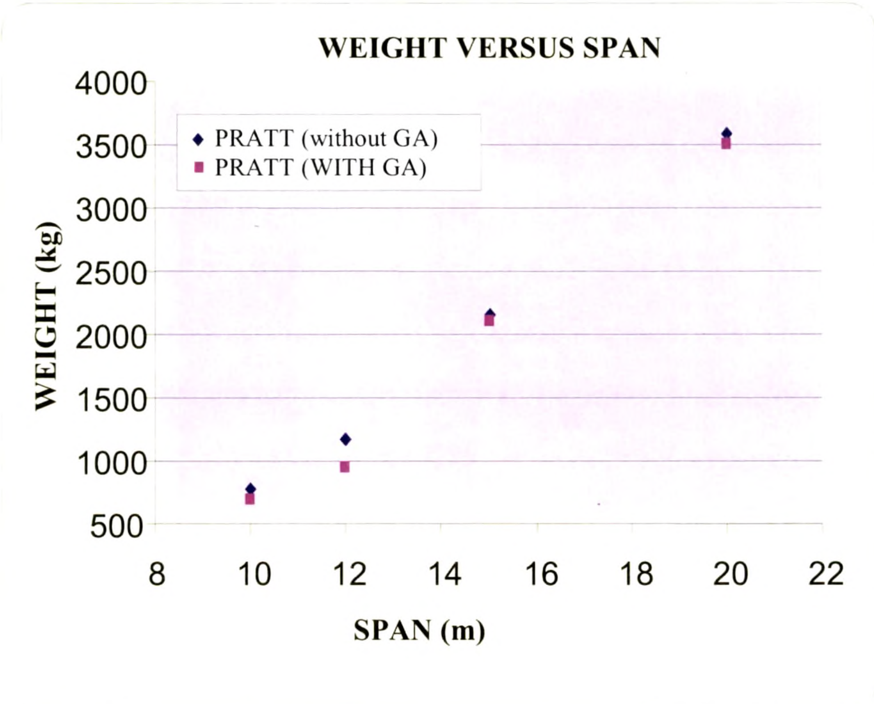

Fig. 10.30 Weight Versus Span Graph for Pratt Truss